DESIGN OF A COAL YARD OF CONCRETE CONSTRUCTION

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

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Design of a coal yard of
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OF A

COAL YARD OF CONCRETE CONSTRUCTION

A THESIS

PRESENTED BY
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V.E.JOHNSON.

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This being the age of Concrete Construction and many beautiful dwellings and factory buildings having been built of concrete, the idea was conceived of designing a modern coal yard of such construction. While there are modern yards having some parts built of concrete, we believe our idea of entire concrete construction to be original.

Since coal bins must be designed to withstand any stresses due to the pressure of the coal stored within them, we are of the opinion that plain concrete block construction is hardly suitable and would certainly not be economical.

We believe that the best structure would be one built of reinforced concrete, the various sections properly designed. Though such a storage shed could not be built as cheaply as one of ordinary timber construction, a brief consideration of the two types will serve to show the economy of the former. A timber shed would require yearly expenses for repairs and painting from the time it was first constructed, and these would tend to increase rather than decrease in amount. It would be necessary to set aside an annual sum for insurance charges and the general depreciation of the building would also have to be taken into consideration if its true value was to be determined each year.

With a properly constructed concrete shed, yearly expenses for repairs, painting and insurance charges would practically be eliminated and there would be no necessity to figure a depreciation charge on the structure.

If the relative costs of the two types are compared at the end of ten years, it will be found that the money saved by the concrete structure in that time will more than make up for the increased first cost over frame construction.

CONCRETE.

Concrete is an artificial stone, and if it contains no steel, that is, if it is not reinforced, it is brittle like stone. Just as stone can be used to support enormous loads, as in foundations, bridges and dams, provided it is so placed as to receive no tension or pull, so can concrete stand heavy loading in compression with no reinforcement.

When well designed and properly constructed, a reinforced concrete structure will be safe for all time, since its strength increases with age, the concrete growing harder and the bond with the steel becoming stronger.

While steel and wooden structures grow weaker from rust and decay a concrete structure as stated above grows stronger with time and its life is measured by ages rather than by years.

In addition to its permanence and strength, concrete is especially suited to the construction of warehouses, terminal buildings, bridges, stations, coal-pockets and similar structures on account of its undoubtable fire-resisting qualities. Actual fires and fire tests have demonstrated time and again the ability of reinforced concrete to withstand even extraordinary fires. This is a valuable asset not only for buildings and warehouses, but particularly for structures to be used for the storage of coal, since the railroads of this country have suffered in the past much inconvenience and expense through the use of inferior bins of timber or steel. The spontaneous combustion to which coal is subject when stored in great quantities not only results in the loss of the coal itself and the damaging of much valuable machinery, but also in the destruction of the bin if it is constructed either of wood or steel.

In compliance then with the above, this design will furnish concrete construction in all parts where there is a possibility of a fire by combustion. The roofing, which will be several feet above the maximum height of the coal at any time, will be of steel construction, the design of which will be found in another part of the book.

The Spontaneous Combustion of Coal.

It was formerly thought that iron pyrites (FeS) was the principal factor in the development of heat in piles of coal. The fact that it has been present where spontaneous combustion occured led to this theory but recent investigations have shown that it has little or nothing to do with it. The argument that overcomes such a theory is as follows:- When moisture and FeS are both present in a pile of coal they tend to enter into chemical composition. This action takes place between the oxygen in the water and the sulphur in the pyrites. However any heat that may be developed will be more than overcome by the remaining water present. Some other theory must therefore be investigated.

Results of long research show that spontaneous ignition is caused; first by absorption of oxygen by coal, second oxidation of coal itself which liberates heat. As iron slowly combines with oxygen resulting in rust so coal oxidizes and loses part of its beneficial properties. The great difference between the two is that if this slow combustion is allowed to proceed the rate of oxidation will increase, which action is contrary to that of iron. As the rate of absorption of oxygen increases in a geometric ratio with the rise of temperature, the conditions for spontaneous combustion are produced by a certain initial temperature, an absorption of oxygen, followed by oxidation. Again the tendency toward spontaneous combustion increases with the initial temperature and the rate of absorption of oxygen. The rate of absorption of oxygen increases with the temperature and with the surface exposed to the air. This action need not be feared on the surface coal when in large lumps, as radiation will overcome any such effect, but the more coal is broken up the more susceptible it is to spontaneous combustion. A property of coal is that it is a very poor conductor of heat so when closely packed as is fine coal, combustion is not so much to be feared. Fine dust scattered over the surface however lends itself to rapid absorption, rapid oxidation and finally combustion. Hence clean coal is less liable to action than is dirty coal. It must be considered however that there is a certain grinding action in the lower strata in a pile of coal due to gravity action on the upper strata even if the dust from the delivery car is neglected.

This grinding action, which is noted in mines where the coal has formed, produces the dust, causes a rise in temperature and fresh surfaces for oxidation which are the fundamentals for spontaneous combustion.

Inspection of the result of a fire due to spontaneous combustion in one of Chicago's large yards showed only the remaining ends of floor timbers while the concrete retaining wall which formed the back of the bin was only slightly discolored. Concrete has stood the test of fire under numerous experiments so its ability will not be doubted here. In case a fire should occur it will only be necessary to empty the bin.

Specifications.

The specifications as noted below were obtained from several books and conform with our idea of how such specifications should be written.

Specifications for Portland-Cement Concrete.

1. Cement: The cement shall be portland, either American or foreign, which will meet the requirements of the standard specifications adopted by the American Society for Testing Materials.
2. Sand: The sand shall be clean, sharp, coarse, and of grains varying in size. It shall be free from sticks and other foreign matter, but it may contain clay or loam not to exceed five per cent. Crusher dust, screened to reject all particles over one quarter inch in diameter, may be used instead of sand, if approved by the engineer.
3. Stone: The stone shall be sound, hard and durable, crushed to sizes not exceeding two inches in any direction. For reinforced concrete, the sizes usually are not to exceed three-quarter inch in any direction, but may be varied to suit the character of the reinforcing material.
4. Gravel: The gravel shall be composed of clean pebbles of hard and durable stone of sizes not exceeding two inches in diameter, free from clay and other impurities except sand. When containing sand in any considerable quantity, the amount per unit of volume of gravel shall be determined accurately to admit of the proper proportion of sand being maintained in the concrete mixture.
5. Water: The water shall be clean and reasonably clear, free from sulphuric acid or strong alkalies.
6. Mixing by Machine: A machine mixer shall be used as the volume of work will justify the expense of installing the plant. The necessary requirements for the machine shall be that a precise and regular proportioning of materials can be controlled, and the product as delivered shall be of the required consistency and thoroughly mixed.
7. Consistency: The concrete shall be of such consistency that when dumped in place it will not require much tamping. It shall be spaded down, and be tamped sufficiently to level it off, after which the water should rise freely to the surface.

8. Forms: (a) Forms shall be well built, substantial and unyielding, properly braced or tied together by means of wire or rods, and shall conform to the lines given.
(b) For all important work, the lumber used for face work shall be dressed on one side and both edges, and shall be sound and free from loose knots, secured to the studding or uprights in horizontal lines.
(c) For backing and other rough work, undressed lumber may be used.
(d) Where corners of the masonry and other projections liable to injury occur, suitable moldings shall be placed in the angles of the forms to round or bevel them off. On edges of piers and in openings in walls we will use curb-bars and angles to prevent concrete from chipping.
(e) Lumber once used in forms shall be cleaned before being used again.
(f) The forms must not be removed within thirty-six hours after all the concrete in that section has been placed. In freezing weather, they must remain until the concrete has had a sufficient time to become thoroughly hardened.
(g) In dry but not freezing weather, the forms shall be drenched with water before the concrete is placed against them.
9. Depositing: (a) Each layer should be left somewhat rough to insure bonding with the next layer above; and, if the concrete has already set, it shall be thoroughly cleaned by scrubbing with coarse brushes and water before the next layer is placed upon it.
(b) Concrete shall be deposited in the moulds in layers of such thickness and position as shall be specified by the engineer in charge. Temporary planking shall be placed at the ends of partial layers, so that none shall run out to a thin edge. In general, excepting in arch work, all concrete must be deposited in horizontal layers of uniform thickness throughout.
(c) The work shall be carried up in sections of convenient length and the sections shall be completed without intermission.
(d) In no case shall work on a section stop within 18 inches of the top.
(e) Concrete shall be placed immediately after mixing, and any having an initial set shall be rejected.
10. Expansion Joints: In exposed work, expansion joints may be provided at intervals of thirty to one hundred feet, as the character of the structure may require (30 feet in our case).

10 (b) A temporary vertical form or partition of plank shall be set up, and the section behind shall be completed as though it were the end of the structure. The partition shall be removed when the next section is begun, and the new concrete shall be placed against the old without mortar flushing.

(c) In reinforced concrete the length of these sections may be materially increased at the option of the engineer.

11. Facing: (a) The facing may be made by carefully working the coarse stone back from the form by means of a shovel, bar or similar tool so as to bring the excess mortar of the concrete.

(b) About one inch of mortar (not grout) of the same proportions as used in the concrete may be placed next to the forms immediately in advance of the concrete in order to secure a perfect face.

(c) Care must be taken to remove from the inside of the forms any dry mortar in order to secure a perfect face.

12. Reinforced Concrete: Where concrete is deposited in connection with metal reinforcing, the greatest care must be taken to insure the coating of the metal with the mortar and the thorough compacting of the concrete around the metal. Whenever it is practicable, the metal shall be placed in position first. This can usually be done where the metal occurs in the bottoms of the forms, by supporting the metal on transverse wires or otherwise, and then flushing the bottoms of the forms with cement mortar, so as to get the mortar under the metal and depositing concrete immediately afterward. The mortar for flushing the bars shall be composed of one part cement and two parts sand. The metal, used in the concrete shall be free from dirt, oil or grease. All mill scale shall be removed by hammering the metal, or by immersing same in a weak solution of muriatic acid. No salt shall be used in reinforced concrete when laid in freezing weather.

Specifications for Laying Vitrified Brick Roadway.

Foundation.

The object of the foundation is to artificially reinforce the prepared natural foundation for like purposes: that is, it shall be made to uniformly sustain the wearing surface, to form a monolithic support to which may be transmitted the weight of the traffic.

Concrete Foundation.

The rock for the concrete foundation should be of hard quality, free from all refuse and foreign matter, with no fragment larger than will pass in its largest dimension through a two (2) inch ring.

Clean, sharp, dry sand shall be thoroughly mixed in its dry state with an approved brand of Portland cement until the whole mass shows an even shade. The proportions of mixture should be one (1) part of cement and three (3) parts of sand.

To this above mixture should be added sufficient clean water to mix to a plastic mass, fluid enough to rapidly subside when attempted to heap into a cone shape.

To this mixture of sand and cement add six (6) parts of damp crushed stone, or good gravel carrying sufficient sand to make the mixture, and then turn the whole mass over not less than three (3) times, or until every fragment is thoroughly coated with the cement mixture. A template spanning the entire width or one-half (1/2) of the street may be used as a guide in leveling the concrete. After thirty-six (36) hours the cushion sand may be spread.

Sand Cushion.

The object of the two (2) inch sand cushion is to afford a uniform support to the monolithic wearing surface - an equilibrium to the vibrations of the impact, yet a relief so slight that the cement bond will not be shattered.

The sand should be practically free from foreign or loamy matter. It need not necessarily be sharp. The cushion should be two (2) inches thick, thoroughly and uniformly compressed before the brick are placed upon it.

The sand should be sufficiently fine so that it will pass through a quarter (1/4) inch mesh. It should be spread by the aid of a template having a steel faced edge, reaching the whole or one-half (1/2) the width of the street and made to conform to the true curvature of the street cross-section.

The compressing should be done with a hand roller weighing from three hundred (300) to four hundred (400) pounds.

Expansion Cushion.

The object of the expansion cushion is to afford a relief to the expansive forces that follow the ranges of temperature in the pavement. When extreme ranges prevail the cushion should be sufficiently wide to relieve the strain.

An expansion cushion must be provided for parallel with and next to the curb. It should be one (1) inch wide for streets thirty (30) feet and less in width, and one and one-half (1 1/2) inches in width for streets wider than thirty (30) feet.

Brick.

The use of vitrified brick for paving is to furnish a durable wearing plate which will have little traction resistance and the greatest amount of wear resistance, and afford a sanitary condition.

They should be first class and thoroughly vitrified, showing at least one fairly straight face, upon which slight kiln marks are allowed. In size they shall not be less than 2 1/2 x 4 x 8 nor more than 3 3/4 x 4 x 9 1/2 inches.

After the brick in the pavement are inspected and surface swept clean, they must be well rolled until each brick is firmly imbedded in the sand cushion.

The Filler.

The object of the filler is to make the pavement monolithic and so unite the brick units that the wear on them will be no more than that of friction and grinding.

The filler shall be composed of one (1) part each of clean, sharp, fine sand and Portland cement. The sand should be dry. The mixture, not exceeding ome (1) sack of the cement, together with a like amount of sand, shall be placed in the box and mixed dry, until the mass assumes an even and unbroken shade. Water shall then be added, forming a liquid mixture of the consistency of thin cream. After the joints are filled flush with the top of the brick and sufficient time for hardening has elapsed, so that the coating of sand will not absorb any moisture from the cement mixture, one-half (1/2) inch of sand shall be spread over the whole surface, and in case the work is subjected to a hot summer sun, an occasional sprinkling, sufficient to dampen the sand, should be followed for two or three days.

Yard Plan and Track Layout.

The yard, Plate (1) of (13), is six hundred (600) feet by one hundred and fifty (150) feet and adjoins the property line of the railroad on its longer side. The elevation of the base of the rail (main track) was assumed to be twelve (12) feet above the proposed finished floor level of the yard. The siding shall consist of two parallel tracks eighty (80) feet on centers, track "A" running parallel to the railroad retaining wall and fifteen (15) feet from it and track "B" running the full length of the yard and connected to track "A" as shown in the plan. If practicable, track "A" may be moved so that its center line may not be less than seventeen (17) feet from the center line of the main track. The track to which "A" connects is for freight service only, the switch point being forty-five (45) feet outside of the north lot line. The connection between this main track and siding "A" is made with a number 6 1/2 frog and a reversed 14 degree 30 minute curve. At a point about ten (10) feet beyond this frog point a turn-out from the outside of the 14 degree 30 minute curve is made which connects with track "B" by means of a number 6 1/2 frog and a reversed 20 degree curve. This is shown on Plate 1.

Another 14 degree 30 minute curve is used to join to track "B". If allowable, we were considering having track "B" elevated a few feet higher than track "A", thus making the switch track an inclined one. The switch point on track "B" is approximately two hundred and seventy-five (275) feet from the south lot line. The tracks are so arranged that 30 cars (car 40 feet long) may be accommodated at any one time.

Pier Design.

Below are the calculations for the standard pier - track "A".

$15\frac{1}{2} \times 4\ 1/2 = 69.75$ square feet area of bottom of pier.

69.75 x 2 = 139.5 cu. ft. volume of footing.
322.5 cu. ft. volume of pier above.
462.0 cu. ft. entire volume.

462 x 150 = 69300# weight of concrete.
4000# weight of stringers.
4500# weight of ties.
1000# rails and fastenings.
132000# Engine load.
210800# total load.

Assume 3500# to be the bearing pressure of soil.

Then $\frac{210800}{3500} = 60.2$ square feet bearing area required.

We have nearly 70 square feet so are safe.

Footing.

Using Professor Talbot's formula for Footings we have:-

$$M = (1/8\ a(l-a)^2 + 3/40(l-a)^3)W$$

where a = 3'-0"
l = 4'-6"
W = 3500#

Therefore M = 30366"#

Assume R = 78

$d = \sqrt{\frac{30366}{12 \times 78}} = 5.7"$ for bending.

Punching Shear.

The upward vertical pressure of the ground on the footing is 15.5 x .75 x 3500 = 40775#
Allowed shear is 40# per square inch.
Allowable shear = 15.5 x 2 x 144 x 40 = 178560# so our assumed depth of 24" is O. K.

Design of Stringers over Roadway.

Dead Load 8" x 12" tie		325#
Rails and fastenings		150#
Guard Rails at 60#'		60#
	Total	535#
Stringer: assume 2- 15" I beams at 90#		90#
	Total	625#

Bending Moment $= \dfrac{625 \times 18 \times 18}{8} = 25313$ '#

Live Load:-

A live load will be assumed for these calculations which is equivalent to a Cooper's E 44 loading. The position for maximum bending moment will be,

Maximum L. L. B. M.

$$\frac{22}{9} < \frac{66}{18} \qquad P_1 < P < P_2$$

$$\frac{44}{9} > \frac{66}{18}$$

$$\therefore\ 2.44 < 3.66 < 4.89$$

B. M. $= \dfrac{66}{2} \times 9 - 22 \times 4.5 = 198000$'#

∴ L. L. B. M. = 198000'#
D. L. B. M. = 25313'#
Allow
33% for impact = 66000'#
20% tractive force 39600'#
Total B. M. = 328913'#

Specifications allow an axial tension on a net section of 16000 pounds per square inch. The effective depth in this section will be the full depth of 15". Therefore the required net section area equals:-

$$\frac{328913 \times 12}{15 \times 16000} = 16.45 \text{ square inches.}$$

Assume

2-15"-45# Channels	24.96 sq."
t .62	
Deduct 4 holes 1" diameter	2.50
	22.46 sq. "

Size of bolts and rivets to be used throughout will be 7/8" diameter.

Design for shear:-
Dead Load shear $\approx 9 \times 650 = 5850\#$
Maximum live load shear:-
Position of load

Moments about R_2

$$\frac{22(18+13.5+9+4.5)}{18} \approx 55000\#$$

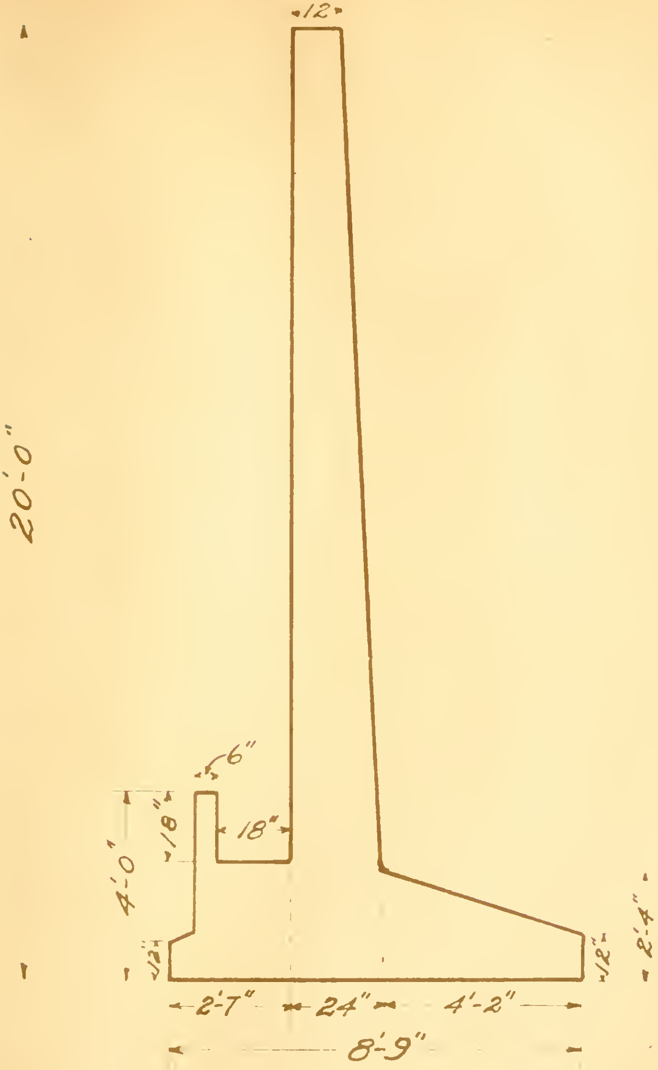
Total shear $\approx 5850 + 55000 = 60850\#$

Specifications allow 10000# per square inch.
The required net area then is $60850 \div 10000 = 6.08$ sq. "

Design for B. M. will govern in this case and the section will be used as assumed.

The wood stringers used are 16" as has been stated before. The difference in depth will be made up by connection plates as shown on the drawings appearing at the end of this work.

MAIN RETAINING WALL



Main Retaining Wall.

In the design of this wall we assumed a height of 20'-0", figuring 4'-0" below ground level and 16'-0" above. Below is the general data used in the design of this wall:-

-Data-

Height above ground level to be 16'-0"
Depth below ground level to be 4'-0"
Maximum allowable pressure of the earth is 4000# per square foot.
Coefficient of friction of concrete on earth is .4
Assume coal to weigh 58# per cubic foot.
Assume concrete to weigh 150# per cubic foot.
Width of wall at top is 12"
Width of wall at bottom is 2'-0"
Total width of base of wall is 8'-9" (see sketch on preceeding sheet).

Working stresses of steel is 15000#

Working stress of concrete is 600#

Shear is 40# per square inch.

Bond stress not to exceed 110# per square inch.

This design is to be investigated in all cases for Bending Momemnt, Shear and Bond Stress.

It was necessary to resort to the ellipse of stress in designing this wall and the following sheet will explain how the ellipse was constructed.

Ellipse of Stress.

In looking through many books on the subject of reinforced concrete retaining walls, we were unable to find any author who had designed a wall for holding coal. We were, therefore, unable to use a comparative design and had to resort to the ellipse of stress.

The ellipse of stress was constructed for a depth of ten (10) feet, the construction being as follows:- Assuming coal to weigh 58# per cubic foot, the vertical pressure at a depth of ten (10) feet is 580pounds. This amount was laid off vertically to scale as shown by line OL (drawing 4 of 13). We next lay off line OT indefinite in length making an angle with OL equal to the angle of repose of coal (30degrees for our case). At the point L draw a horizontal line until it intersects OT at B. With B as a center and radius BL draw the arc of a circle so that it will be tangent to line OL at L. The angle TBL is equal to 2θ . Bisect this angle by the line BG. Now at O draw a line through O parallel to BG and with a radius OT and O as a center strike an arc until it cuts the line OK at K. The distance OK is equal to value P. Now at O draw the line CC' parallel to the back of the wall and draw ON normal to the line CC'. On line ON lay off the distance OB equal to $\frac{P+Q}{2}$. With any convenient radius strike an arc using N as a center and then on this arc lay off twice the angle that OK makes with the normal ON. Through the intersection of the arcs draw the line NM which will equal $\underline{P-Q}$. The points L, K and M will be points on the ellipse.

Perpendicular to the line OK draw the line SOS' and with O as a center and OV as a radius strike the arc at S and S' . The line OS and OS' is equal in value to Q . The line OM will be the resultant stress in direction and intensity. A smooth line drawn through the points S, L, K, M, S' will give the required ellipse.

The weight of the wall is determined by dividing it into sections and finding the weight of each section. The sections are taken to be 12" wide.

Section (1) $1 \times 20 \times 150\#$ equals 3000#

Section (2) $1/2 \times 10 \times 150\#$ equals 750#

Section (3) $\frac{10 + 20}{2 \times 12} \times 2.58 \times 150\#$ equals 485#

Section (4) $\frac{12 + 28}{2 \times 12} \times 4.16 \times 150\#$ equals 1037#

Section (5) $\left\{\frac{16 + 19}{2 \times 12} \times 2 \quad \frac{6 \times 1}{12}\right\} \times 150\#$ equals $\underline{525\#}$

Total concrete load is 5797 or
say 5800#

The weight of the coal and some earth above the base

is 4100#

Therefore total load equals 5800# 4100# equals 9900#

The distance to the center of gravity of the whole mass is found by multiplying the weight of each section by its respective lever arm, all measured from a common point or line. The sum total divided by the total weight is the distance to the center of gravity of the whole mass.
Let X equal the distance to the center of gravity.

$$X \text{ equals } \frac{1037 \times 2.4 + 3750 \times 5.4 + 485 \times 7.3 + 525 \times 7.5 + 4100 \times 2.08}{9900}$$

$$X \text{ equals } \frac{38723}{9900}$$ equals 3.91 feet from the extreme right

hand end of base. All of this is shown on plate 4 of 13.

Sliding and Overturning (Graphical)

$\frac{5.6}{2}$ equals 2.8 so wall is safe against sliding.

$\frac{14.6}{2}$ equals 7.3 so wall is safe against overturning.

Stress at Heel and Toe of Wall.

Use the formula $\frac{P}{A}\left(1 \pm \frac{6\ e}{L}\right)$

Where A equals area of the base

L equals length of base in feet.

e equals the eccentricity.

P equals vertical component of the resultant.

e equals .2

L equals 8.74

P equals 10500

Therefore $\frac{10500}{8.74}\left(1 \pm \frac{6 \times .2}{8.74}\right)$ equals $1200(1 \pm .14)$

this gives 1370# for Toe

and 1030# for Heel.

A stress diagram is shown on plate 4 of 13.

Bending Moment, Shear and Bond Stress.

Section (1) at base (see plate 4 of 13)
Thickness equals 1.9'
Lever arm scaled equals 3.45'
B. M. equals 5306 x 3.45 x 12 equals 220000"#

$f_c = 600$ $f_s = 15000$ $n = 15$ $p = .0075$ $R = 98$

M equals Rbd^2 assume b equal 12" for all calculations.

$220000 = 98 \times 12 \times d^2$

$d^2 = 187$

$d = 13.7$ say 14"

Shear:-

Allowable shear is 40# per square inch.
Area of section is 12 x 23 = 276 square inches.
Horizontal component of R = 4600# (5306 x .866)

$\frac{4600}{276}$ equals 16.7# therefore o. k. for shear.

Steel:-

$$A = \frac{M}{f_s jd}$$

or $\frac{220000}{15000 \times .875 \times 23}$ equals .73 square "

Use $\frac{5}{8}$ inch round rods spaced 5" c. c.

$$u = \frac{V}{\Sigma_o jd} = \frac{4600}{2 \times 1.96 \times .875 \times 23} = 58\#$$ o.k. for bond stress.

Section (2) 5'-0" above section (1)

Thickness equals 1.67'
Lever arm scaled equals 2.0'
B. M. equals 2025 x 2 x 12 equals 48600"#

M equals Rbd^2

$48600 = 98 \times 12 \times d^2$

$d^2 = 41.5$

$d = 6.45$ say 8" at least.

Shear:-
Area of section = 12 x 20 = 240 square "
Horizontal component of R equals 1750#

$\frac{1750}{240}$ equals 7.3 o.k. for shear.

Steel:-

$$A = \frac{48600}{15000 \times .875 \times 17}$$ equals .218 say .22 square "

Use 1/2 " round rods spaced 10" c.c.

Bond Stress:-

$$u = \frac{1750}{2 \times 1.57 \times .875 \times 17}$$ equals 37# o.k. for bond stress.

Section (3) 10'-0" above section (1)

Thickness equals 1.41'
Lever arm scaled equals .55'
B. M. = 310 x .55 x 12 equals 2045"#

M equals Rbd^2

$2045 = 98 \times 12 \times d^2$
$d^2 = 1.74$
$d = 1.32$ say 2"

Shear:-
Area of section = 12 x 17 = 204 square "
Horizontal component of R = 270#

$\frac{270}{204}$ equals 1.3# for shear therefore o.k.

Steel:-

$$A = \frac{2045}{15000 \times .875 \times 14}$$ equals .0111 square"

Use 1/2 " round rods spaced 20" o.c. as a greater spacing is impracticable.

Bond Stress:-

$$u = \frac{270}{1.57 \times .875 \times 14}$$ equals 14# o.k. for bond stress.

Design of Toe.

Bending:-

$$\frac{1370 + 1270}{2} \times 2.58$$ equals 3410#

B.M. equals 3410 x 1.3 x 12 equals 53150"#

$M = Rbd^2$

$53150 = 98 \times 12 \times d^2$

$d^2 = 45.2$

$d = 6.74"$

Shear:-

$\frac{3410}{20x12}$ equals 14.3# which is less than 40# o.k.

Steel:-

$A = \frac{M}{f_s jd} = \frac{53150}{15000x.875x17}$ equals .19 square"

Use 3/8" round rods spaced 6" c.c.

Bond Stress:-

$u = \frac{3410}{2x1.5x.875x17}$ equals 77# therefore o.k.

Design of Heel.

Bending:-

$\frac{1190 + 1030}{2}$ x 4.16 equals 4630#

B. M. equals 4630 x 2 x 12 equals 111000"#

$111000 = 98 \times 12 \times d^2$

$d^2 = 92.6$

$d = 9.64"$ therefore o.k.

Shear:-

$\frac{4630}{28x12}$ equals 13.8# which is less than 40# o. k.

Steel:-

$A = \frac{111000}{15000x.875x25}$ equals .30 square"

Use 7/16" round rods spaced 6" c.c.
Bond stress figured to be 78# therefore o.k.

Temperature Steel.

Per cent of steel in stem is .2%

Per cent of steel in base is .1%

Area of stem is (18.2+18.2x1/2)144 equals 3930 square"

3930 x .002 equals 7.86 square"

$\frac{7.86}{18.2}$ equals .432 square"

Use 3/4" round rods spaced 12" c.c.

Area of base equals (8.74 x 1.25) 144 equals 1570 square"

1570 x .001 equals 1.57 square"

$\frac{1.57}{8.74}$ equals .18 square"

Use 1/2" round rods spaced 12" c.c.

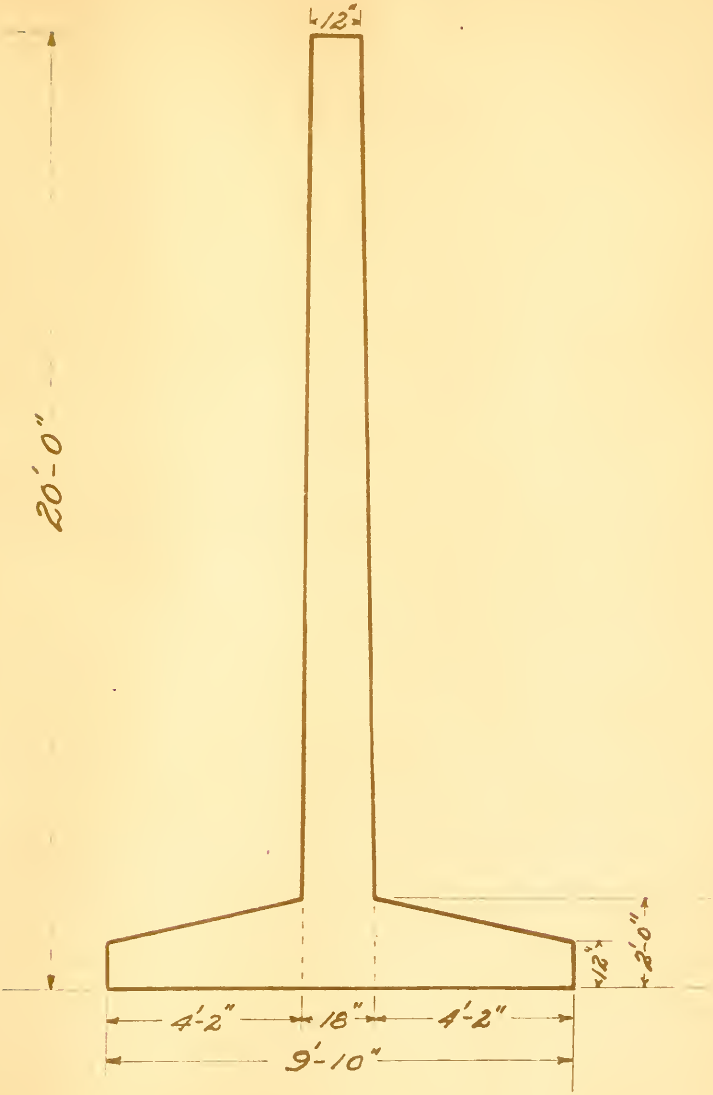
Instead of using 1/2" and 3/4" rods for temperature steel we will use all 3/4" rods.

As will be seen, the dimensions of the drawing and of the design do not compare very closely. It was suggested to us that the minimum width of wall at top should be twelve (12) inches and at least one face be battered, as shown in the drawing.

It is our idea to place concrete pockets at different places along this wall and in this case the wall would have to act as a column. We are, therefore, setting aside our figured design and using our assumed values in the design of the main retaining wall.

At every twenty-four (24) feet along this main wall we are to have an opening to allow the coal to run into the conveyor. This will be shown on the main drawing.

BIN PARTITION WALL



Design of Bin Partition Wall.

This wall will be closely patterned after the main retaining wall. The pressures will be the same as those figured in the previous case but there are a few minor changes in dimensions all of which will be checked in the following design. These changes in dimensions will be noted on the sketch as shown on the previous page.

Weight of wall:-

Stem $\frac{1+1.5}{2} \times 18$ equals 22.5

Base 2×1.5 equals 3.0

$2 \times \frac{1+2}{2} \times 4.16$ equals $\underline{12.48}$

37.98

37.98 x 150# equals 5700# the weight of concrete.

The condition considered in this design is when a maximum load is behind one face of the wall and the other bin is empty.

Distance to Center of Gravity.

Weight of retaining wall equals 5700# and is applied at the vertical axis of the wall.
Weight of coal and earth behind wall equals 4100#.

$$X = \frac{4100 \times 2.08 + 5700 \times 4.92}{9800} = \frac{8528 + 28044}{9800}$$

X = 3.73 feet from right hand end of base.

Graphical determination for sliding and overturning.

$\frac{11.9}{2.7}$ equals 4.41 o.k. for sliding.

$\frac{25.2}{2.7}$ equals 9.34 o.k. for overturning.

Stress at Heel and Toe of Wall.

Eccentricity equals .58 feet.
Vertical component of R equals 10400#

Use the formula $\frac{P}{A}\left(1 \pm \frac{6e}{L}\right)$

Where A equals the area of the base.
L equals the length of the base.
e equals the eccentricity.
P equals the vertical component of the resultant.

Therefore $\frac{10400}{9.83}\left(1 \pm \frac{6 \times .58}{9.83}\right)$ equals 1060 $(1 \pm .35)$

this gives 1430# for Heel
and 690# for Toe.

Bending Moment, Shear and Bond Stress.

Section (1) at base, see plate 6 of 13.

Thickness equals 1.5 feet.

From the ellipse of stress and other drawing we have:-

B.M. equals 5306 x 3.45 x 12 equals 220000"#

$M = Rbd^2$

$220000 = 98 \times 12\ d^2$

$d = 14"$ therefore o.k.

Shear:-
Area of section equals 12 x 18 equals 216 square "
Horizontal component of R equals 4600#
$\frac{4600}{216}$ equals 21.3# o.k. for shear.

Steel for Section (1)

A equals $\frac{220000}{15000 \times .875 \times 15}$ equals 1.12 square"

Use 3/4 " square rods spaced 6" c.c.

Bond Stress:-

u equals $\frac{4600}{2 \times 3.00 \times .875 \times 15}$ equals 59# o.k. for bond.

Section (2)

Thickness equals 16.32 "

B.M. equals 2025 x 2 x 12 equals 48600"#

$48600 = 98 \times 12 d^2$

d = 7" therefore o.k.

Shear:-
Area of section equals 16.32 x 12 equals 196 square"
Horizontal component of R equals 1750#

$\frac{1750}{196}$ equals 8.93# o.k. for shear.

Steel:-

A equals $\frac{48600}{15000 \times .875 \times 14}$ equals .27 square "

Use 3/4 " square rods spaced 12" c.c.

Bond Stress:-

u equals $\frac{1750}{2 \times 1.96 \times .875 \times 14}$ equals 37# o.k. for bond.

Section (3)

Thickness equals 14.66 "
B.M. equals 310 x .55 x 12 equals 2045 "#
$2045 = 98 \times 12 d^2$
d = 2" therefore o.k.

Section (3) Shear:-
Area of section equals 14.66 x 12 equals 176 square"
Horizontal component of R equals 270#

$$\frac{270}{176} \text{ equals } 1.54\# \text{ o.k. for shear.}$$

Steel:-

$$A \text{ equals } \frac{2045}{15000x.875x12} \text{ equals } .011 \text{ square"}$$

Use 3/4" square rods spaced 24" c.c.

In order to have a uniform design we shall use 3/4" square rods spaced 24" c.c. although this spacing is far less than actually required but 24 " is about the maximum spacing.

Bond Stress:-

$$u \text{ equals } \frac{270}{1.96x.875x12} \text{ equals } 13.2\# \text{ o.k. for bond.}$$

Section (4)

The rods of this section will be the same as those designed for section (3) as a change in design would be impracticable and this section of wall will take only compression.

Design of Toe.

Bending:-

$$\frac{690+1000}{2} \text{ x } 4.16 \text{ equals } 3520\#$$

B.M. equals 3520 x 2 x 12 equals 84500"#

$84500 = 98x\ 12d^2$

$d^2 = 71.9$

d = 8.5 " therefore o.k.

Shear:-

Area of section equals 12 x 24 equals 288 square"
Horizontal component of R equals 3520#

$\frac{3520}{288}$ equals 12.3# o.k. for shear.

Steel:-

A equals $\frac{84500}{15000 \times .875 \times 21}$ equals .31 square"

Use 7/16" square rods spaced 6" c.c.

Bond Stress:-

u equals $\frac{3520}{2 \times 1.75 \times .875 \times 21}$ equals 18.3# o.k. for bond

Design of Heel.

Bending equals $\frac{1430 + 1100}{2}$ x 4.16 equals 5275#

B.M. equals 5275 x 2.2 x 12 equals 139400"#

$139400 = 98 \times 12d^2$

$d^2 = 118.7$

$d = 10.9"$ o.k.

Shear:-

Area of section equals 288 square"

$\frac{5275}{288}$ equals 18.3# o.k. for shear.

Steel:-

A equals $\frac{139,400}{15000 \times .875 \times 21}$ equals .51 square"

Use 5/8" round rods spaced 6" c.c.
Bond stress figured to be 13.7# therefore o.k.
Temperature rods are to be the same as in other wall.

Design of Wall Between Piers.

The design of this wall will be made with the assumption that it will act as a cantilever and we will allow 40% for impact. The condition of maximum stress exists when one side is fully loaded and the other side empty. The loads as computed by means of the ellipse of stress are used here.

Pressure at base equals 1060#
40% impact allowed 424#
1484#

$$\text{B.M. equals } \frac{12}{8} \times 1484 \times \left(\frac{15}{2}\right)^2 \text{ equals } 125000\text{"\#}$$

125000 equals $98 \times 12d^2$

$d = 10.4" + 1\ 1/2"$(fireproofing) say 12"

Shear V equals 7.5 x 1484 equals 11130#

$$\text{Unit shear v equals } \frac{11130}{12\ x} \text{ equals } 40$$

Solving for x we get 16.2" and on account of size required this will be assumed to govern.

Steel:-

$$\text{A equals } \frac{125{,}000}{15000 \times .875 \times 16.2} \text{ equals .59 square"}$$

Use 1/2" square rods spaced 4" o.c. as the required area is not sufficient for required bond stress.

Bond Stress:-

$$\text{u equals } \frac{11130}{3 \times 2 \times .875 \times 16.5} \text{ equals 110\# o.K.}$$

This design applies to the section of the wall to a point 3'-0" above the floor line.

Design of section (2)

Pressure at 5'-0" section equals 640#
Impact 40% equals 256#
896#

B.M. equals $\frac{12}{8} \times 896 \times \left(\frac{15}{2}\right)^2$ equals 75940"#

75940 equals 98 x $12d^2$

$d = 8.05" + 1\ 1/2"$ fireproofing say 9 1/2"

Shear V equals 7.5 x 896 equals 6720#

Unit shear v equals $\frac{6720}{12\ x}$ equals 40#

Solving for x we get 5.6" therefore B.M. governs.

Steel:-

$$A \text{ equals } \frac{75940}{15000 \times .875 \times 8.05} \text{ equals } .72 \text{ sq."}$$

Use 1/2" square rods spaced 4" c.c.

Bond Stress:-

$$u \text{ equals } \frac{6720}{4 \times 2 \times .875 \times 8.05} \text{ equals } 120\#$$

This is too large for bond stress but the section as actually drawn is thicker so bond stress is decreased.

Design of section (3)

Pressure at 10'-0" section equals 340#
Impact 40% equals 136#
476#

B.M. equals $\frac{12}{8} \times 476 \times \left(\frac{15}{2}\right)^2$ equals 40500"#

$40500 = 98 \times 12d^2$
$d = 5.9"$

Section (3) Shear:-

Shear V equals 7.5 x 476 equals 3580#

$$\text{Unit shear v equals } \frac{3580}{12\ x} \text{ equals } 40\#$$

Solving for x we get 7.5" say 12" as a less thickness would be impracticable.

Steel:-

$$A \text{ equals } \frac{40500}{15000 \times .875 \times 10.5} \text{ equals } .294 \text{ square "}$$

Use 1/2" round rods spaced 8" o.c.

Bond Stress:-

$$u \text{ equals } \frac{3580}{3 \times 1.57 \times .875 \times 10.5} \text{ equals } 83\# \text{ bond o.k.}$$

The thickness of section (2) using 18" base and 12" top is 16". The bond stress will then be:-

$$u \text{ equals } \frac{6720}{3 \times 2.0 \times .875 \times 14.5} \text{ equals } 89\# \text{ o.k.}$$

The remaining height of this partition wall will have the same steel as was designed for section (3) The section was made 12" at the top on account of the additional wearing action of the coal, and the top corners will be protected with a corner bar.

COAL POCKET TYPE -A-

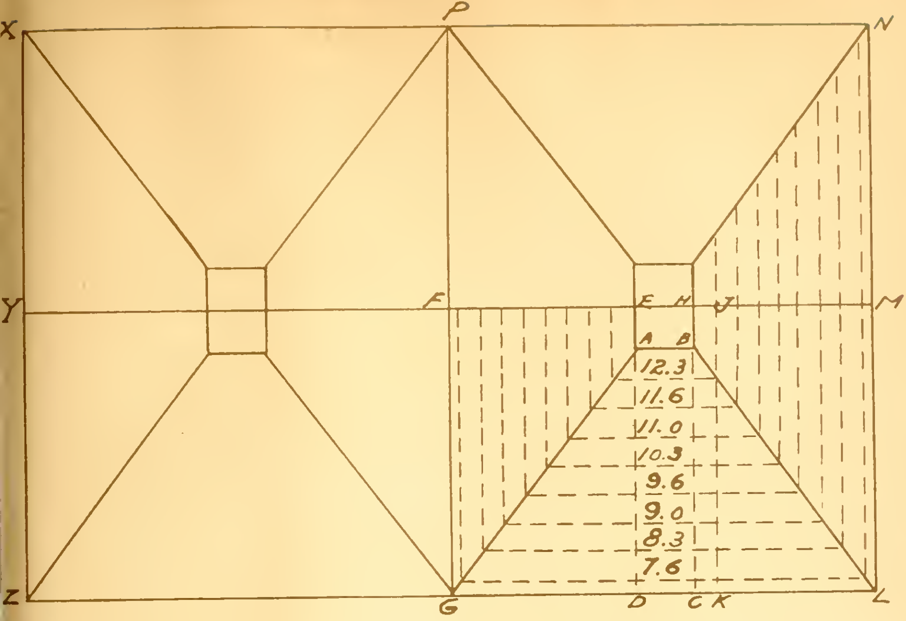


FIGURE 1

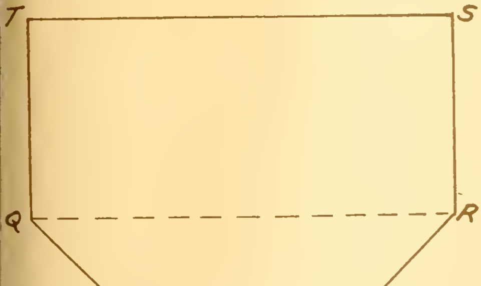


FIGURE 3

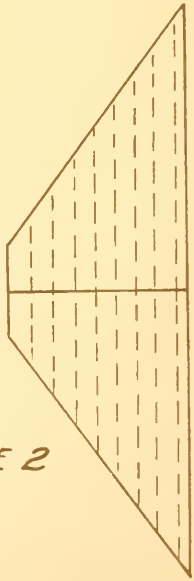


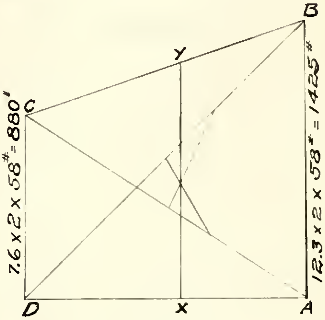
FIGURE 2

Design of Pocket Type A

The coal pockets are similar in design except that type A has four (4) compartments while type B has only two (2). All the necessary information in regard to dimensions, reinforcing and sections can be obtained by referring to drawings 8 and 9 of 13.

Type A

Consider the section ABCD.
The loading is determined by assuming sections whose projections are one foot wide. This gives the following loading for this section.



Consider dead load at 100# sq foot.
The average load on this section is found by scaling the line XY which is 20.3 x 58# equals 1180#

Load on section equals $\frac{1180}{2}$ 100 equals 690#

B.M. equals $\frac{12}{8}$ x 690 x $(8.5)^2$ equals 75000"#
75000 equals 98 x $12d^2$
d = 8"–

Reaction at AB for 1'-0" section:-

$R = (7.6x.9 + 8.3x1.8 + 9x2.8 + 9.6x3.8 + 10.3x4.9 + 11x5.9 + 11.6x6.9 + 12.3x7.9) \div 8.5$

R = 44.25x58 + 8'-6"x10/12x150 equals 2566 + 1064 equals 3630#

$$d = \frac{3630}{12x40} = x = 7.56"$$

Therefore thickness is governed by B.M., requiring 10"

Steel:-

$$A \text{ equals } \frac{75000}{15000x.875x8.0} \text{ equals } .72 \text{ sq."}$$

Use 1/2" square rods spaced 4" c.c.

Bond Stress:-

$$u \text{ equals } \frac{3630}{3x\ 2\ x.875x\ 8.0} \text{ equals } 86\# \text{ o.k. for bond.}$$

Consider section AB as a beam taking the reaction of 3630# per foot figured above:-

$$M. \text{ equals } \frac{12}{8} x\ 3630\ x\ (3)^2 \text{ equals } 49000"\#$$

$49000 = 98 x 12d^2$

d = 6.44"

Shear equals 3630#

$$d = \frac{3630}{12x40} \text{ equals } 7.56"$$

Section will not change for this beam action.

Steel:-

$$A \text{ equals } \frac{49000}{15000\ x\ .875\ x\ 8} \text{ equals } .47 \text{sq."}$$

This steel is to be placed parallel to AB and on account of conditions which will exist at this point an excess will be used for safety.
Use 1/2" square rods spaced 3" o.c. A equals 1.0 sq."

Bond Stress:-

$$u \text{ equals } \frac{3630}{3 \times 2 \times .875 \times 8} \text{ equals } 87\# \text{ so o.k.}$$

Required Stirrups:-

$v_1 = 87-40 = 47\#$

$S = \frac{d}{4} = 3"$

$P_s = v_1 bs = 47 \times 12 \times 3 = 1700\#$

$f_s = 12000\#$

$$A \text{ equals } \frac{1700}{12000} \text{ equals } .14 \text{ sq. "}$$

Use 3/8" square spaced 3" o.c.

Design of Steel for Section HJKC.

According to pressure diagram shown on page 32,

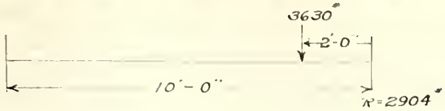
the average load carried to AB is 690#

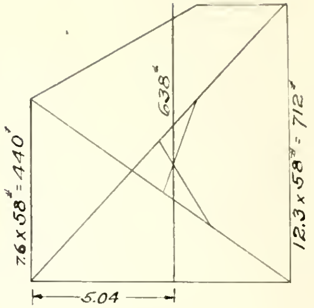
$$\text{Reaction is } \left(44.25 + \frac{12.3 \times 9.3}{8.5}\right)53 = 3347\#$$

Average load figured from diagram (2) is 640+160 for D.L. gives a total of 800#

$$M = \frac{12}{8} \times 800 \times (10)^2 = 120000"\#$$

Moment due to concentrated load at B





M equals 2904 x 24" equals 69700"#

M equals 69700 + 120000 equals 189700"#

189700 equals 98 x 12 d^2

d equals 16.1"

Reaction figured above equals	3347
Reaction due to conc. load $\frac{4}{5}$ x 3630 equals	2903
Dead Load 10'-0" x 150 equals	1500
Total reaction equals	7750#

Shear:-

$$d \text{ equals } \frac{7750}{12 \times 40} \text{ equals } 16.1"$$

A beam section will be used along all such lines as H, J, K, C. The depth will be 18" and width 12".

Steel:-

$$A \text{ equals } \frac{189700}{15000 \times .875 \times 16.1} \text{ equals } .90\text{sq.}"$$

Use 3 - 9/16" sq. rods.

Bond Stress:-

$$u \text{ equals } \frac{7750}{3 \times 2.25 \times .875 \times 16.1} \text{ equals } 82\#$$

v_1 equals 82-40 equals 42#

S equals $\frac{d}{4}$ equals 3"

P_s equals $v_1 bs$ equals 42 x 12 x 3 equals 1510#

f_s equals 12000#

A equals $\frac{1510}{12000}$ equals .13 sq ".

For stirrups use 3/8" square spaced 3" o.c.

The remainder of section HMLC will be of slab construction. The loading will be determined by the following figure:-

L. L. equals 638#
D. L. equals 102#
740#

M equals $\frac{12}{8}$ x 740 x $(10)^2$ equals 111000"#

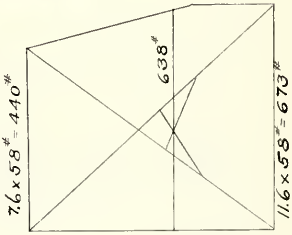
111000 equals 93 x 12 d^2

d equals 9.8"

Shear:-

V equals 740 x 5 equals 3700#

d equals $\frac{3700}{12 \times 40}$ equals 7.7"



The thickness of the section is therefore 10" to agree with the balance of the pocket floor.

Steel:-

A equals $\frac{111000}{15000 \times .875 \times 9.8}$ equals .87sq. "

Use 1/2" square rods spaced 3 1/2 " o.c.

Bond Stress:-

$$u \text{ equals } \frac{3700}{3 \times 2 \times .875 \times 9.8} \text{ equals } 72\# \text{ o.k.}$$

Temperature Steel:-

$$p \text{ equals } \frac{.86}{11 \times 12} \text{ equals } .0065$$

As this is excessive .4% will be used.

Projected section is 11" x 10'-0" equals 1320 sq".

1320 x .004 equals 5.28 sq".

$$\text{Area per section equals } \frac{5.28}{10} \text{ equals } .53 \text{ sq}"$$

Use 1/2" square rods spaced 6" c.c. o.k.

Test of Bearing Under Pocket.

Loading (maximum)

$$\text{Reaction of beam PG equals } \frac{169700}{2} \text{ equals } 84850\#$$

The beam is 16" wide.

$$\text{Therefore load per foot of wall is } \frac{84850}{1.33} \text{ equals } 60000\#$$

Weight of wall and fill is 9900#

69900#

$$\text{Required bearing area is } \frac{69900}{4000} \text{ equals } 17.5 \text{ sq}'.$$

Area of base of wall is 8.75 sq ft. per foot of section. This action is safe to be considered as acting out at an angle of 30 degrees. The wall is 20'-0" high so that the area then of the base of the vertical section alone is 11.5 x 2 equals 23 sq. ft. which is greater than 17.5 No additional material is therefore necessary to take care of this concentrated loading.

Design of Pocket Walls.

The pressures will be assumed from diagram 4 of 13

Design of section QRST of this wall.
Pressure at base 7'-0" depth is 205#
Consider as a beam uniformly loaded.

$$M \text{ equals } \frac{12}{12} \times 410 \times (10)^2 \text{ equals } 41000''\#$$

$$41000 \text{ equals } 98 \times 12\, d^2$$

d equals 5.9"

Shear:-

$$V \text{ equals } 410 \times 5 \text{ equals } 2050\ \#$$

$$v \text{ equals } \frac{2050}{12 \times 40} \text{ equals } 4.3''$$

Total thickness then is 8"

Steel:-

$$A \text{ equals } \frac{41000}{15000 \times .875 \times 4.2} \text{ equals } .76 \text{ sq }''.$$

Use 5/8" square rods spaced 6" o.c.

Bond Stress:-

$$u \text{ equals } \frac{2050}{2 \times 2.5 \times .875 \times 5.9} \text{ equals } 80\#$$

This section may have to be increased due to the required span of 20'-0" between walls. It is therefore not necessary to figure for another section of reduced area above that just designed. On account of the small remaining section the required steel will not be changed.

Design of vertical section below line OR — of the pocket — for lateral pressure.
From the ellipse of stress this pressure is 725#

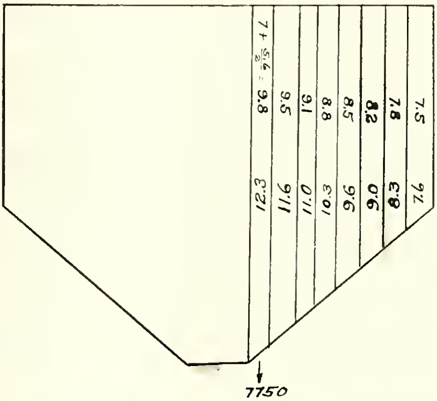
M equals $\frac{12}{12} \times 725 \left(\frac{15}{2}\right)^2$ equals 40750"#

40750 equals 98 x 12 d^2

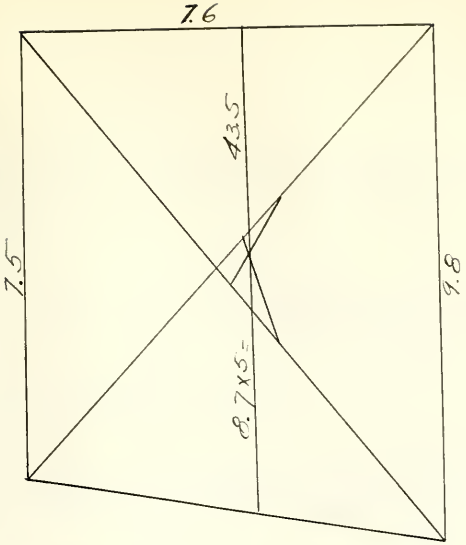
d equals 5.9"

This indicates that the same section figured above will be on the side of safety for these trapezoidal sections.

Section designed for vertical loading. Beam MFY



Section loading shown on diagram above, figured by triangle method on figure 2. These are then average unit loads and must be multiplied by 1/2 L or 5'-0" to give the reaction for which this section must be designed. This gives the following diagram from which the average load is calculated.



Average reaction equals 43.5 x 58# equals 2523#

M equals $\frac{12}{8}$ x 2523 x $(15)^2$ equals 851500"#

M due to concentrated load of beam equals 7750 x 6.1 equals 472750"#

Assume dead load of 200# per sq. ft. of surface. Surface of trapezoidal section is 5.6 x 11.5 equals 64.5 sq. ft.

Average equals $\frac{64.5}{15}$ equals 4.3+7 equals 11.3 for whole wall.

Average dead load equals 11.3 x 200 equals 2260#

M due to D.L. is 12/12 x 2260 x $(15)^2$ equals	507500
M due to floor load in pocket is	851500
ditto same	851500
M due to concentrated load is	472750
ditto same	472750
	3156000

M equals f_s A j d.

d equals depth of pocket on vertical though it should be measured perpendicularly to the steel.

Steel:-

$$A \text{ equals } \frac{3156000}{15000 \text{x} .875 \text{x} 12 \text{x} \left(7 + \frac{5.6}{2}\right)} \text{ equals } 2.04 \text{ sq.}''$$

Use 2 -- 1 1/8" square rods.

$$\frac{2M}{kjbd^2} \text{ equals } f_c \text{ equals } \frac{2 \text{ x } 3156000}{.375 \text{x} .875 \text{x} 12 \text{x} (117.6)^2} \text{ equals } 116$$

Shear:-

Due to D.L. equals 7.5x2260 equals	16900
Due to floor load is 7.5 x 2523 equals	18900
ditto same	18900
Concentrated load is	7750
ditto same	7750
	70200#

Bond Stress:-

$$u \text{ equals } \frac{70200}{2 \text{ x } 4.5 \text{ x } .875 \text{ x } 117.6} \text{ equals } 76\#$$

Design of Beam PG

Load due to D.L. (b equals 16" and d is 7'-0")

1.33 x 20 x 7 x 150 equals	29300#
Reaction figured for beam FM	70200
ditto same	70200
	140400#

D. L. Mom. is 12/8 x 29300 x 20 is	879000"#
Conc. L. Mom. is 12/4 x 140400x20 is	8424000"#
	9303000"#

Steel:-

$$A \text{ equals } \frac{9303000}{15000 \times .875 \times 80} \text{ equals } 8.87 \text{ sq."}$$

Use 4 — 1 1/2 " square rods.

Shear:-

$$D.L. \text{ equals } \frac{29300}{2} \text{ equals } 14650\#$$

$$L.L. \text{ equals } \frac{140400}{2} \text{ equals } 70200\#$$

$$\overline{84850\#}$$

$$d \text{ equals } \frac{84850}{16 \times 40} \text{ equals } 133"$$

The section is thus found to be much too small so stirrups will be used.

Bond Stress:-

$$u \text{ equals } \frac{84850}{4 \times 6 \times .875 \times 80} \text{ equals } 50\#$$

Required stirrups:-

$$V \text{ equals } \frac{84850}{16 \times 80} \text{ equals } 67\#$$

67-40 equals 27#

Assume s equals 16"

P_s equals 27x16x16 equals 6900#
f_s equals 12000

$$A \text{ equals } \frac{6900}{12000} \text{ equals } .58 \text{sq."}$$

Use 7/8" round rods spaced 16" o.c.

Design of Beam LN.

This beam will have the same dimensions as beam PG but the loading requires revised beam steel.

Loading:-

Due to D. L. equals 1.33 x 7 x 150 x 20 is	29300
Shear figured for beam FM is	70200
Total	99500#

D.L.Mom. is 12/8 x 29300 x 20 is	879000"#
Conc. load Mom. is 12/4 x 70200x20 is	4212000"#
	5091000"#

Steel:-

$$A \text{ equals } \frac{5091000}{15000 \times .875 \times 80} \text{ equals } 4.85 \text{ sq.}''$$

Use 4 — 1 1/4" round rods.

Shear:-

$$\text{D.L. is } \frac{29300}{2} \text{ equals } 14650$$

$$\text{L.L. is } \frac{70200}{2} \text{ equals } \underline{35100}$$

$$49750\#$$

$$V \text{ equals } \frac{49750}{16 \times 80} \text{ equals } 41.5\#$$

Bond Stress:-

$$u \text{ equals } \frac{49750}{4 \times 3.93 \times .875 \times 80} \text{ equals } 46\#$$

The amount of shear in excess of that required is so small it could be neglected but on account of the possibility of unsymmetrical loading the minimum size stirrups will be used at a spacing of 18". The size will be 3/8" square rods.

Design of Pocket —Type B.

This type will be identical with MYZL of Type A in plan. This requires that columns must support the outer side. These occur at points Y, F and M. The following is a summation of the design.

Section ABCD. Number of these 2
Thickness 10"
Steel required 1/2" sq. rods spaced 4" c.c. 12'-0" long.

Beam AB No.of these 2
Thickness 10"
Steel required 4—1/2" sq. rods. 4'-0" long.
3/8" sq. stirrups spaced 3" c.c. 2'-6" long.

Beam HJKC No. of these 4
Thickness 18"
Steel required 3 9/16" sq. rods 40Ft. long.
3/8" sq. stirrups spaced 3" c.c.

Alternate on this section.
Imbedded 7" - 15# I beam 3 pieces with connections.
2 pieces at 10'-6"
1 piece at 4'-6"

Section JMLK
Thickness 10"
Steel- 1/2" sq. rods spaced 3 1/2" c.c. varying lengths.

Temperature steel all sides use 1/2" sq. rods spaced 12"

Sides:- LG, GZ, ML, FG, & YZ.
Thickness 8".
Use 5/8" sq. rods spaced 6" c.c.
A double row of steel in partition FG will be necessary on account of the pressure from either side.

Side MFY.
Thickness 12"
Horizontal steel use 5/8" sq. rods spaced 6" c.c.
Beam steel use 2 7/8" sq. rods.

Design of MFY will be found on following page.

Design of MFY.
See page 39 for loading.

Average L.L. reaction is 43.5 x 58 is 2523#

M is $\frac{12}{8} \times 2523 \times (15)^2$ or 851500"#
M due to reaction of HJKC is 7750 x 6.1 or 472750"#
M due to D.L. is $\frac{12}{12} \times 2260 \times (15)^2$ or 507500"#
Total M 1831750"#

Area for steel is $\frac{1831750}{15000 \times .875 \times 12 \left(7.0 + \frac{5.6}{2}\right)}$ is 1.43 sq."

Use 2 7/8" sq. rods.
Assume b is 12"

f_c is $\frac{2 \times 1831750}{.375 \times .875 \times 12 \times (117.6)^2}$ is 68#

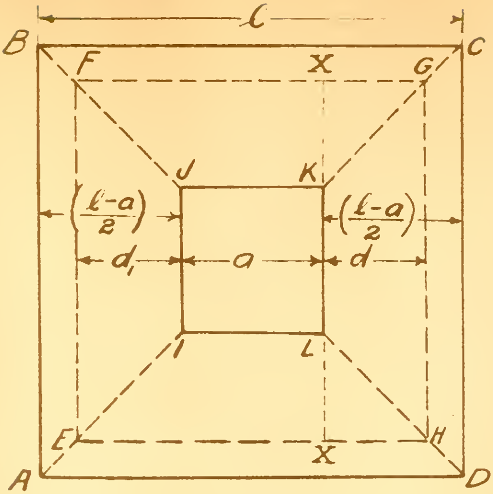
Shear:-
Due to D.L. is 7.5 x 2260 is 16900#
Due to L.L. is 7.5 x 2523 is 18900#
Due to beam reaction is 7750#
43550#

Bond Stress:-

u is $\frac{43550}{2 \times 3.5 \times .875 \times 117.6}$ is 61#

V is $\frac{43550}{12 \times 117.6}$ is 30.9# which is less than 40# o.k.

FOOTING DESIGN.



ℓ
B
C
F
X
G
J
K
$\left(\frac{\ell-a}{2}\right)$
$\left(\frac{\ell-a}{2}\right)$
d_1
a
d
I
L
E
X
H
A
D

BENDING MOMENT TO BE CALCULATED ALONG LINE XX AT FACE OF COLUMN.

PUNCHING SHEAR TO BE CALCULATED AT I-J-K-L FACES OF COLUMN.

DIAGONAL TENSION CALCULATED ALONG LINE E-F-G-H. DISTANT "d" INCHES FROM FACE OF COLUMN.

BENDING MOMENT CALCULATED BY CONSIDERING GROUND LOAD ON TRAPEZOID C-K-L-D ABOUT XX.

$$M = \left[\tfrac{1}{8}a(\ell-a)^2 + \tfrac{3}{40}(\ell-a)^3\right]w$$

PUNCHING SHEAR:

$$V_p = (\ell^2 - a^2)w$$

$$v_p = \frac{V_p}{4ajd} = \frac{(\ell^2-a^2)w}{4ajd}$$

DIAGONAL TENSION.

$$V_{dt} = [\ell^2 - (a+2d)^2]w$$

$$v_{dt} = \frac{V_{dt}}{4(a+2d)jd}$$

BOND STRESS FIGURE ON LINE XX.

$$\mu = \frac{V_b}{\Sigma_o jd}$$

where $V_b = \frac{1}{4}(\ell^2 - a^2)w$

Σ_o = SUM OF PERIMETERS OF BARS.

Columns.

Three columns are required to support type B pocket, one size for M and Y, and a larger section for F.

Load on column:-

D.L. from LM is $\frac{5+7}{2} \times 2/3 \times 150$	600#
D.L. column assuming 12"x12" column	300#
Reaction from FM	43550#
Roof load assume 40# per sq.ft.40x7.5x5	1500#
Total load	45950#

These columns have an effective height of 16'-0" The steel section to support this load of 46000# is by (Carnegie 1913 edition)

4 angles 3 1/2"x 2 1/2"x 5/16"
1 plate 8" x 1/4"

This gives a column size of 12"x12" when encased in concrete.

Design of Column F

Load:-

D.L. is	600#
D.L. of column is	300#
1/2 of FM and 1/2 of FY is	87100#
Roof load 40x5x15 is	3000#
Total load	91000#

Effective height is 16'-0"

From same table in Carnegie the section is

4 angles 4" x 3" x 3/8"
1 plate 8"x5/16"

Design of Column Footing.

Load:-

D.L. from previous page is 91000#
Size of column is 12"x12".

Required area of base is $\frac{91000}{4000}$ is 23 sq.feet.

Assume D.L. of base as $\frac{12000}{4000}$ or $\underline{3}$ sq. feet.

26 sq.feet.

Size of base therefore is 5'-2" by 5'-2"

(1-a) is (5'-2" - 1') or 4'-2"

M is equal to $w\left\{1/8a(1-a)^2 + 3/40(1-a)^3\right\}$

equals $4000(1/8(5.17-1)^2 + 3/40(5.17-1)^3)12$

equals 365760"#

V_p equals $(1^2 - a^2)w$

or$((5.17)^2 - (1)^2)4000$ or 102800#

d equals $\frac{V_p}{4ajv_p}$ where v_p is unit shear not greater than 120# allowed by Chioago Building Ordinance.

d equals $\frac{102800}{4x12x.875x120}$ equals 20.4"

b equals a + 2d + 1/2(1-a-2d)
or 12 + 40.8 + 1/2(62-12-40.8) equals 57.4"

d equals $\sqrt{\frac{M}{Rb}}$ equals $\sqrt{\frac{365760}{98 \times 57.4}}$ equals 8.1"

Therefore depth for punching shear governs.

Required Steel:-

$$A \text{ equals } \frac{365760}{15000 x .875 x 20.4} \text{ equals } 1.37 \text{sq.}"$$

Use 7 1/2" sq.rods.

Bond Stress:-

$$u \text{ equals } \frac{102800}{4 x 14.0 x .875 x 20.4} \text{ equals } 103\# \text{ o.k.}$$

Considering diagonal tension.

$$V_{dt} \text{ equals } (l^2 - (a + 2d)^2)w$$

$$V_{dt} \text{ equals } (\overline{5.17}^2 - \left\{\frac{12 + 20.4}{12}\right\}^2)4000$$

$$\text{equals } 134000\#$$

$$d \text{ equals } \frac{V_{dt}}{40(a + 2d)j v_{dt}}$$

v_{dt} allowed by Chicago Building Ordinance is 40#

$$d \text{ equals } \frac{134000}{4\ (12 + 40.8)\ x .875\ x\ 40} \text{ equals } 18.1"$$

Concrete bearing area required is $\frac{91000}{400}$ or 227 sq.feet.

Make the footing a block 5'-2" x 5'-2" and 2'-0" high.

For the column base and cap design see plate 9 of 13

The design of column footing#2 is similar to the one just worked but on account of the changes in size of footing and new steel we will show our calculations.

Design of Column Footing #2

D.L. as in previous case is 46000#

Required area for column load is $\frac{46000}{4000}$ or 11.5sq.ft.

Assume D.L. footing equal $\frac{8000}{4000}$ or 2.0sq.ft.

Assume size of base to be 3'-9" square.

1-a equals 3'-9" minus 1'-0" equals 2'-9"

$$M \text{ equals } 4000\left\{1/8(3.75-1')^2 + 3/40(3.75-1')^3\right\}12$$

equals 120240"#

V_p equals $(3.75)^2 - (1)^2$ x4000 equals 60240#

$$d \text{ equals } \frac{60240}{4x12x.875x120} \text{ equals } 12"$$

b equals 12 + 24 + 1/2(45-12-24) equals 40.5"

$$d \text{ equals } \sqrt{\frac{120240}{98x40.5}} \text{ equals } 5.4"$$

Therefore depth for punching shear governs.

Steel:-

$$A \text{ equals } \frac{120240}{15000x.875x12} \text{ equals } .77 \text{ sq."}$$

Use 5 - 5/8" sq. rods.

Bond Stress:-

$$u \text{ equals } \frac{60240}{4.12.5x.875x12} \text{ equals } 115\# \text{ o.k.}$$

Considering diagonal tension.

$$V_{dt} \text{ equals } \left(\overline{3.75}^{2} - \left(\frac{12+12}{12}\right)^{2}\right)4000$$

$$\text{equals } 40240\#$$

$$d \text{ equals } \frac{40240}{40(12+24) \times .875 \times 40} \text{ equals } 8''$$

$$\text{Concrete bearing required equals } \frac{46000}{400} \text{ equals } 115\text{sq.}''$$

Therefore make the footing a concrete block 3'-9" square and 12" high. For the column base and cap design see plate 9 of 13.

Alternate to beam construction HJKC.

This beam may be omitted if a steel I beam can be inserted so that equal strength is developed. Such a construction will greatly simplify the form work.

M equals $S\frac{I}{C}$

The moment figured from a preceeding page was 189700"#

Therefore $\frac{189700}{18000}$ equals 10.52

Use a 7" 15# I beam.

On account of the doubtful result of bending, the following design will be used. The section neglects any effect of the surrounding concrete.

A sketch on this sheet is not necessary as a detailed drawing is shown on plate 6

Consider the splice plates shown on the above figure as taking the entire shear and the cover-plates to take the moment.
Shear as has been figured was 7750#
Using 5/8" rivets the required number is only 3. Five will be used however with the minimum thickness of plate i.e. 3/8" thick on each side of the web.

Cover Plates.

The cover plates are to take the full bending moment.
Make section moment equal to moment of assumed member.

Moment of Inertia of section equals 36.2 $''^{4}$

Moment of plates equals M' equals $2ar^{2}$

M' equal to or greater than 36.2

Assume 7/16" plate.

36.2 less than $2(3.66-.38)7/16(3.5+.22)^{2}$

36.2 less than 39.8

This considers one plate on top and one on the lower side, the latter being in tension and therefore will have subtracted from its area that due to rivet holes. Rivets in single shear.

Required rivets equals $\frac{16000\times3.28\times7/16}{2\times3068}$ equals 4

Required length of plate is 2x3x3 equals 18".

The rivets will then be placed in each flange as shown below in the figure.

See plate 6 for detailed drawing.

÷TABLE "A"÷

MEMBER	STRESS	SECTION	RIVETS	LENGTH.
BI	0	1L 2½×2×¼	3	3.6'
BH	-7840	2Ls 3×2×5/16	7	8.6'
CH	+3250	1L 2½×2×¼	3	7.2'
GC	-9740	2Ls 3½×2½×¼	5	10.6'
DG	+6600	1L 2½×2×¼	3	10.8'
FD	-12260	2Ls 4×3×5/16	6	13.35'
EF	-11520	2Ls 5×3½×3/8	8	14.4'
AB	-19460	2Ls 3×2×5/16	5	8.6'
CB	-12820	" "	5	8.6'
DC	-6125	" "	5	8.6'
ED	+ 600	" "	5	8.6'
IJ	+21500	" "	7	7.85'
HI	+21500	" "	7	7.85'
GH	+14410	" "	7	7.85'
FG	+7180	" "	7	7.85'
MNK	+4840	1L 2½×2×¼	3	7.4'
LK	-6420	2Ls 3×2×5/16	5	8.2'
ML	-7530	" "	5	8.2'

Design of Roof Truss for Hard Coal Bins.

The hard coal bins are the only bins to be covered as the burning properties of hard coal seem to be affected if the coal is exposed to the weather.
The following data is given for the design of the truss.

Span of truss is 35'-6"

Trusses spaced 15'-0" c. to c.

There are five equal panels; the purlins to be placed at panel points only.

Specifications used are C. E. Schneiders.

Loading.

The roofing chosen was composition roofing, weighing 10 pounds per square foot. Weight of snow was taken from the specifications as 18# per square foot. The normal wind load specified is 20# per square foot.

Weight of purlins from the specifications is

W equals $\sqrt{\frac{PD}{45} - 1/4}$; where P is the load per square foot on the purlins and D is the distance between trusses. Substituting these values, W is 3.75 From a similar formula, W equals $\frac{Pl}{300 + bL + PD/3}$

W, or the weight of truss per square foot was found to be 2.1 pounds.

Then the total load is equal to the sum of these; or the steady load is 33.85# per square foot and the wind load 20# per square foot; normal to the rafter.
Using these values the stresses were obtained graphically and tabulated in Table A see previous page.

Design of Purlins.

The bending moment for a beam uniformly loaded and fixed at both ends is M equals 1/8wl where W is the uniform load normal to the rafter.

Normal component of uniform load is
33.85 x cos a equals 33.85x.9 or 30.46
Wind load is 20

$$\begin{array}{r} 30.46 \\ \underline{20} \\ 50.46 \end{array}$$

These values were slightly excessive so 50# was taken as the uniform load per foot. The distance between the purlins is 8.6feet.

Therefore M equals $\dfrac{50 \text{x} 8.6 \text{x} (15)^2 \text{x} 12}{8}$ is 145125 "#

M equals $\dfrac{SI}{C}$ or $\dfrac{I}{C}$ equals $\dfrac{M}{S}$ equals $\dfrac{145125}{16000}$ or 9.07

the section modulus.

An 8" -- 13.75# channel has a section modulus of 9.00,

so this section will be used for all purlins.

Tension Members

Article 48-Specifications states that the length of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

Article 34:- Tension, net section rolled steel is 16000#

Tension member Bottom chord.

Maximum stress is 21500# and the unsupported length is 95"

$\dfrac{21500}{16000}$ equals 1.344"

Two angles 3x2x5/16" have an area of 2.94 square inches.

Since we are using 5/8" rivets the area to be deducted to obtain the net area is 2(5/8 1/8) x 5/16" or .46" Net area is 2.48 sq. inches which is greater than 1.344

Least radius of gyration is $\frac{95}{90}$ is 105 so since this section has sufficient area and its ratio of l/r is less than 200 we will use it.

Tension Member DG

Maximum stress is 6600# and the unsupported length is 172"

$\frac{6600}{16000}$ equals .413"

One angle 2 1/2x2x1/4 has an area of 1.06 sq. inches.
Deduct one hole at .19 is .19
.87 net section.

Use this also for CH, BI, & MNK.

Compression Members

Article 47 of the specifications states that the effective length of main compression members shall not exceed 125 times their least radius of gyration.

Article 39:- Axial compression on gross section of column is $16000 - \frac{701}{R}$ with a maximum of 14000#

Compression Member AB

Stress is 19460# Length is 103"
Assume P at 10000#

Assumed area is $\frac{19460}{10000}$ equals 1.946

Two angles have an area of 2.94 (3x2x5/16")
Least radius of gyration is .85

$\frac{L}{r}$ is $\frac{103}{.85}$ equals 121

P from table 3 is 9770 so this section will be used.

Member DF

Try 2 angles 4x3x5/16 Area is 4.18

Least radius of gyration is 1.27

$$\frac{70L}{r} \text{ is } \frac{70x160}{127} \text{ is } \frac{1120}{127} \text{ is } 8818$$

16000-8818 is 7182

7.182x4.18 is greater than 12260
125 x 1.27 is approximately equal to 160" so this section will be used for DF.

Member EF.

Stress is 11520# and length is 173"
Two angles 5x3x3/8" have an area of 6.10"
Least radius is 1.44

$$\frac{70L}{r} \text{ is } \frac{70x173}{144} \text{ is } 8410$$

16000-8410 is 7590 and 7590x6.10 is greater than 11520

125 x 1.44" is greater than 173" so a section of two angles 5x3 1/2x3/8" will be used.

Member BH

Stress is 7840 and length is 103"
Two angles 3x3x5/16" have an area of 2.94
Least radius is .86

$$16000 - \frac{70L}{r} \text{ is } 16000 - \frac{7210}{86} \text{ is } 3384\#$$

8384x2.94 is greater than 7840
125x.86 is greater than 103" so this section will be used.

Member CG

Maximum stress is 9740 and length is 127"
Two angles 3 1/2 x 2 1/2 x 1/4" have an area of 2.88
Least radius of gyration is 1.06

$$16000 - \frac{70L}{r} \text{ is } 16000 - \frac{70x127}{1.06} \text{ or } \frac{8890}{100} \text{ is } 8390$$

8390 x 2.88 is greater than 9740
125 x 1.06 is greater than 127 so this section will be used.

Member ML

Maximum stress is 7530 and length is 98"

Two angles 3x2x5/16" have an area of 2.94"sq.
Least radius of gyration is .87

$$16000 - \frac{70L}{r} \text{ is } 16000 - \frac{6860}{.87} \text{ is } 8115$$

8115 x 2.94 is greater than 7530

.87 x 125 is greater than 98" so this section will be used.
Also use this section for member LK.

Rivets.

Article 79 of the specifications states that the minimum distance between rivet holes equals 3 times the diameter of rivets or 3 x 5/8 equals 1 7/8" But if possible this distance should not be less than 2" A minimum spacing of 2 1/2" is used throughout the truss. The maximum pitch in the line of stress shall be 4 1/2" for 5/8" rivets. 5/8" rivets and 3/8" gusset plates are used.

Rivets in Top Chord.

Strength is 9770x2.94 or 28724

$$\frac{28724}{5630} \text{ is 5 shop rivets.}$$

Members BI, CH & DG.

Strength is .87 x 16000or 13920

$$\frac{13920}{5630} \text{ is 3 rivets}$$

Rivets in Members HB-LK-ML

As these members have the same section as the top chord we will use 5 rivets.

Member GC

Strength =8390x2.88=24163

$$\frac{24163}{5630} = 5 \text{ rivets.}$$

Member FD

Strength = 7182 x 4.18 = 30000

$$\frac{30000}{5630} = 6 \text{ rivets.}$$

Member EF

Strength = 7590 x 6.10 = 46300

$$\frac{46300}{5630} = 9 \text{ rivets but stress is 1/4 strength so less rivets.}$$

Member MNK

Strength = .87 x 16000 = 13920

$$\frac{13920}{5630} = 3 \text{ rivets.}$$

Lateral Bracing.

Round bars 7/8" in diameter will be used for lateral bracing.

Article 40 of the specifications states that the working stresses for compression or tension may be increased to 20000#

A x 20000 = .60 x 20000 = 12000# tension.

Shear on pin = 2pid/4 x 12000 = 12000

d = .7937

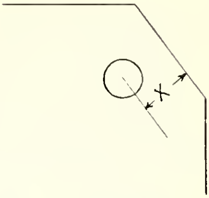
Assume lateral plates 5/16"thick.

Bearing on pin = 5/16 x d x 24000 = 12000

d = 1 3/4"

From page 320 of Cambria a 1 3/4" pin calls for a 3 1/2" head on clevis.

There is a possibility of tear or split when a hole is punched too near an edge as shown in the figure.
(1.6x5/16x X)12000=12000
x=2.09
For split (x-1)5/16 x 16000 = 12000
x = 1.41 for split.
Shearing value for rivet = 3680
12000
----- = 4 rivets required in plate.
3680



Design of Shoe.

The vertical component of the pressure at the point of support = 11160#
Bottom chord section = 2 angles 3 x 2 x 5/16"
Width of plate = 2 x 2 * 5/16 x 3/8 = 4 and 3/4"
Allowed pressure on concrete walls = 600# per sq. inch.
11160
------ = 18.6 sq. inches.
600

18.6
-------- = 3.9 inches = minimum length of bearing plate.
4.75

Make bearing plate 8" long. There must be sufficient space on either side for anchor bolts. Allow 3" on each side. Therefore the dimensions of the plate will be 8" x 10" x 1/2" thick.
Use 2 anchor bolts . Shear on each equals:-
1/2 x 16000/2 (10 3/4 -2) = 28000# shear on each bolt.
Use 1 and 1/2" anchor bolts.

Design of Pocket Covering.

This roofing has the function of keeping snow, rain and other matter out of the coal pockets and therefore out of the coal, keeping it as clean as possible. The roof will be of steel construction throughout. The roof covering will be of galvanized iron supported by steel beams which in turn are supported by steel columns rising from the top of the pocket walls. Plate 13 shows the typical covering for any section of pocket. Type B will drain out into the roadway and type A will drain to the line over the center of the roadway and thence to the ends of the pocket with a slope of 1/4" per foot.

Loading beam A
Wind load neglected.
Galvanized iron 3 x 7.5 x 10 is 225#

$$\frac{I}{C} \text{ equals } \frac{225x(10)^2x12}{8x16000} \text{ equals } 2.11$$

Assume a 7.50# section.

$$\frac{I}{C} \text{ equals } \frac{7.50x(10)^2x12}{8x16000} \text{ equals } .08$$

Total 2.19

Use a 4" — 7.25# Channel.

Beam between columns.
Loading beam B
Reaction is 5(225 + 7.5) is 1160#

$$M \text{ is } \frac{1160}{2} \text{ x } 7.5 \text{ or } 4350\#$$

$$\frac{I}{C} \text{ equals } \frac{4350x12}{16000} \text{ or } 3.26$$

Assume a 9.50# section.
Using same method as above we find that we need a 5" — 9.00# Channel.

Design of the Post.

Loading.

Reaction of Beam B sustaining load is	1160#
D. L. is 9.00 x 15 or	136#
Reaction of beam B sustaining load is	1125
D. L. is 7.25 x 5 or	36#
Assume D. L. of column as	40#
Total	2497#

Assume height of post as 2'-0"
Loading is 2497 say 2500#
On account of the light loading sufficient riveting room will determine the size section to be used.

We will use 2 - 3" - 4.00# Channels.

See Plate 13 for detailed drawing.

Coal Conveyor.

As will be seen on sheet or plate 5 of 13, a conveyor pit runs along the base or rather is cut into the base of the main retaining wall. There are openings along the wall at intervals of 24 feet through which the coal runs into the conveyor. No design or drawing has been made of this conveyor but it will be of a design to be passed upon by the engineer of construction. This conveyor will be made to conform with the size of the pit and be operated by electricity. It is our idea to run the machinery at night only as that will be ample time to fill the different pockets.

When the conveyor reaches its respective pocket it will travel up to the top of the pocket and then discharge the coal, passing down on the opposite side. At the openings in the walls where the coal runs into the conveyor, a sheet or plate of steel will extend as a chute to allow the coal to drop into the conveyor without clogging the pit.

Tile drains provided for bins and roadways shall be passed upon by the engineer before being placed.

Although no mention of it has been made we will use a 1:2:4 mix in all cases in concrete work. The floors of the bins will be reinforced with 1/2" square rods spaced 24" center to center.

The coal bins will be provided with a mechanical device known as a "drag rake" and this will be used to drag the coal from the trestles, where it is dumped, up to the main retaining wall. It is then allowed to fall into the conveyor and is hoisted into the pockets. The pockets are to be fitted with a mechanical chute to discharge the coal into the wagons or auto trucks.

* * *

We are greatly indebted to the Professors of the Civil Engineering Department of the Armour Institute of Technology for their many suggestions and valuable assistance in compiling this thesis.

* * *

Bibliography.

Ketchem's Walls, Bins and Grain Elevators.

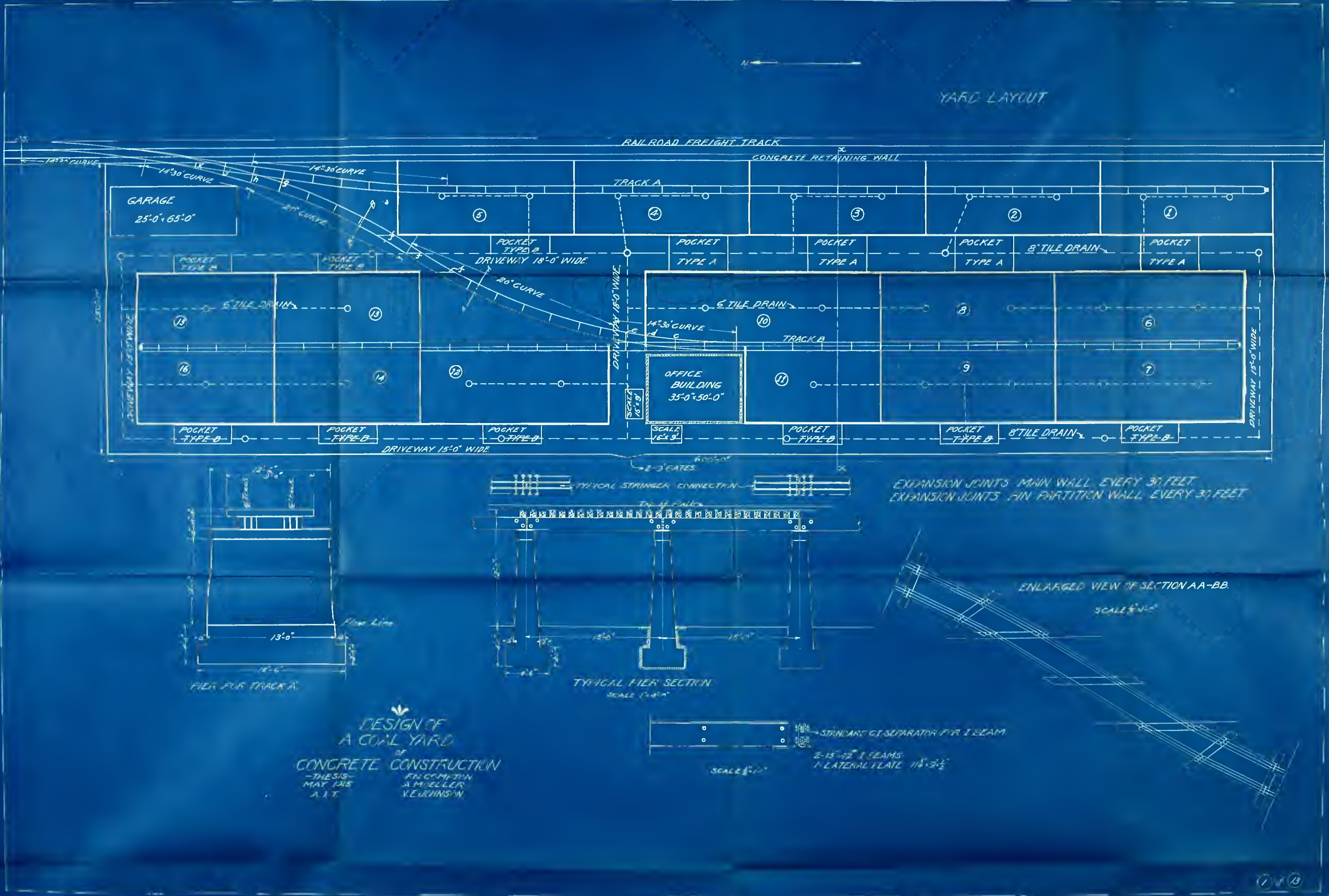
Turneaure & Maurer, Principles of Reinforced Concrete Construction.

Gas World of November 1912, Article by S. F. Walker.

Concrete-Cement Age.

Hand-Books of Universal Portland Cement Company.

Engineering News.



DESIGN OF
A COAL YARD
OF
CONCRETE CONSTRUCTION

-THESIS-
MAY 1915
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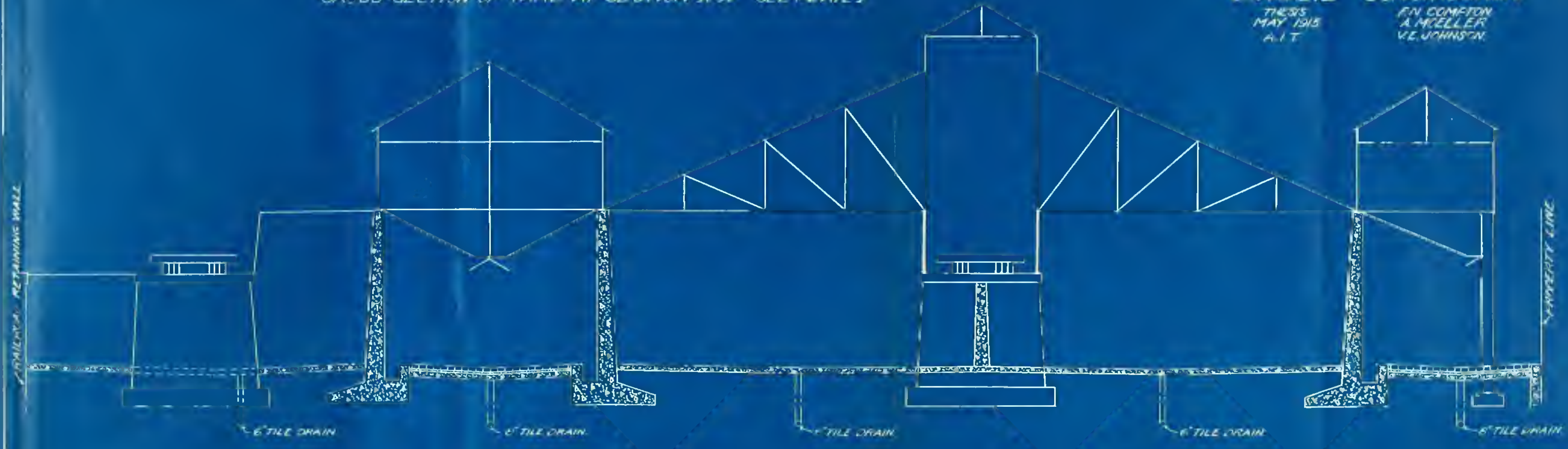
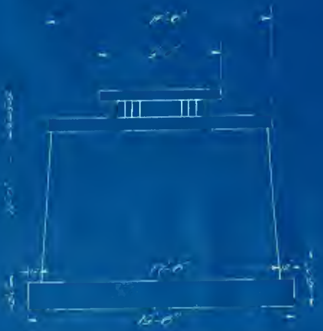
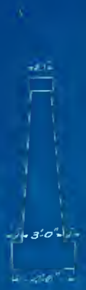
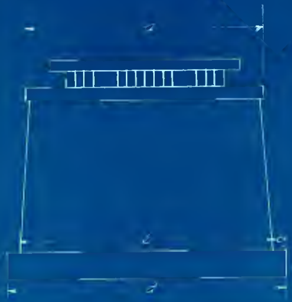
CROSS-SECTION OF YARD AT SECTION X-X. SEE PLATE I





TYPICAL PIER FOR TRACK E
EXCEPT AS NOTED SEE PLATE I





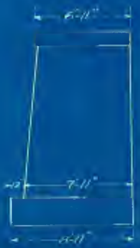
PIER MARKED "C" SEE PLATE 1

CLASSIFICATION	OF	PIERS		
PIERS	a	b	d	c
C	13'-9½"	14'-9½"	16'-9½"	12"
D	15'-9½	16'-9½"	18'-9½"	12"
E	17'-3"	18'-3"	20'-3"	12"
F	17'-3"	18'-3"	20'-3"	12"
G	18'-9½"	19'-9½"	21'-9½"	12"
H	16'-3"	17'-3"	19'-3"	12"
J	14'-3"	15'-3"	16'-3"	12"
K	6'-11"	7'-11"	8'-11"	12"

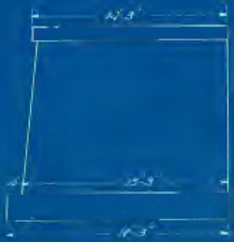
DESIGN OF
A COAL YARD
OF
CONCRETE CONSTRUCTION

THESIS
MAY 1915
A.I.T.

F.N. COMPTON
A. MOELLER
V.E. JOHNSON



PIER MARKED "K" SEE PLATE I

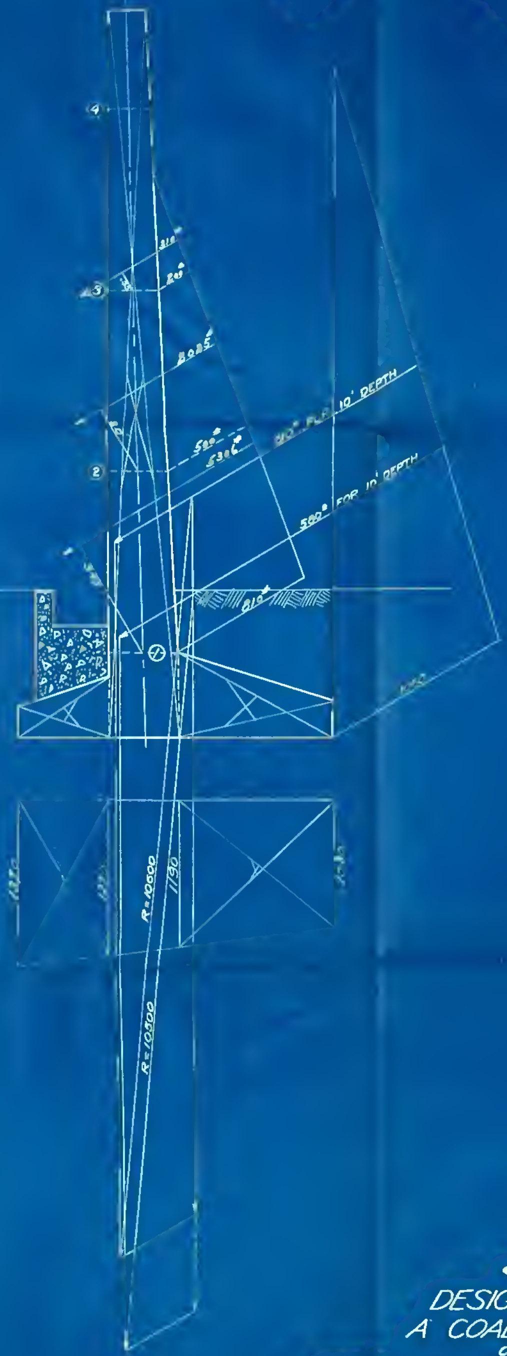


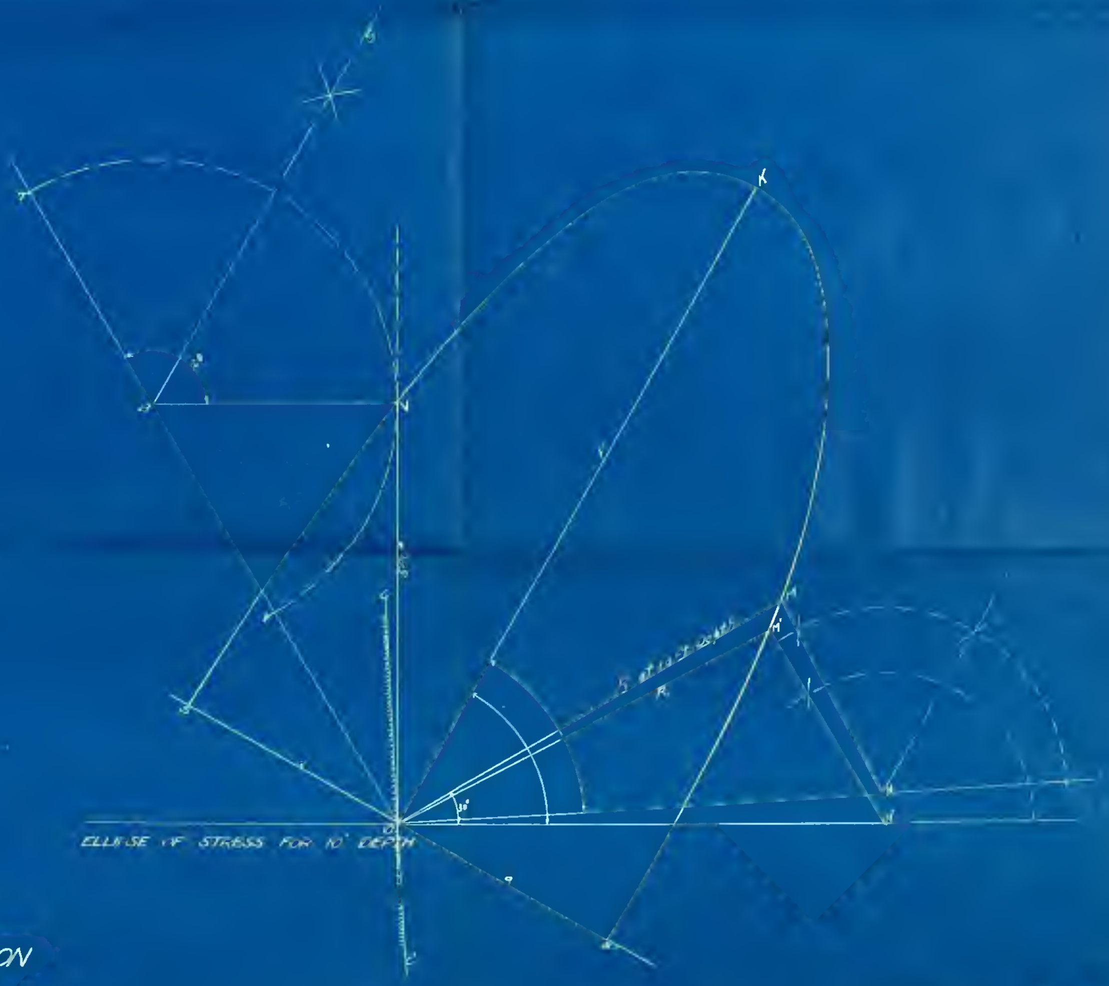
PIER MARKED "J" SEE PLATE I

DETERMINATION OF STRESSES IN MAIN RETAINING WALL

SCALE 1" = 2'

ALL DIMENSIONS AND REENFORCING INDICATED ON





ELLIPSE OF STRESS FOR 10' DEPTH

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CONCRETE CONSTRUCTION

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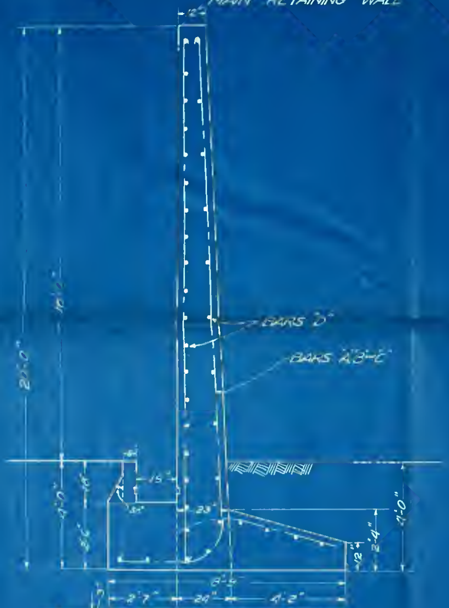
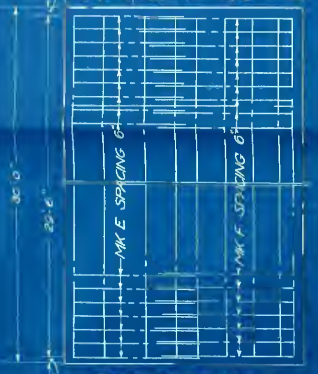
V. E. JOHNSON

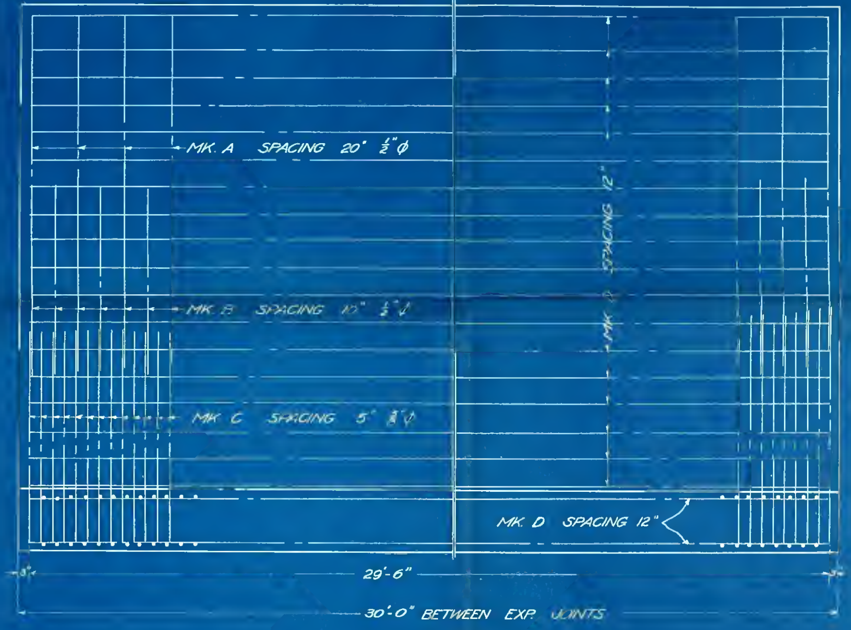
TYPICAL SECTION
MAIN RETAINING WALL





PLAN OF FOOTING

LONGITUDINAL SECTION



BILL OF MATERIAL FOR 30-0" SECTION				
MARK	NO.	SIZE of BARS		REMARKS
A	17	½" ϕ	12'-6"	
B	18	½" ϕ	6'-6"	
C	71	⅝" ϕ	7'-6"	
D	37	¾" ϕ	30'-0"	HOOKED
E	59	⅜" ϕ	6'-0"	
F	59	⅜" ϕ	6'-0"	

NOTES

WEIGHT of CONCRETE 150# PER CU.FT.
" COAL 56# " " "
NO SURCHARGE
FOUNDATION CLAY

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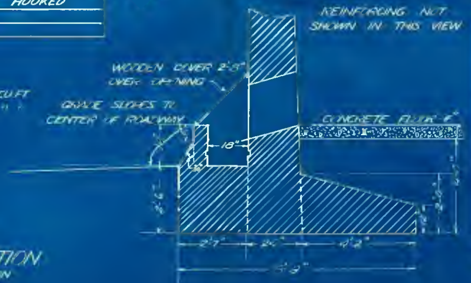
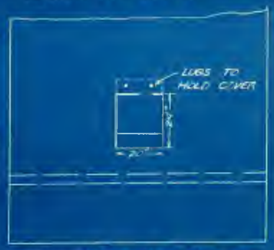
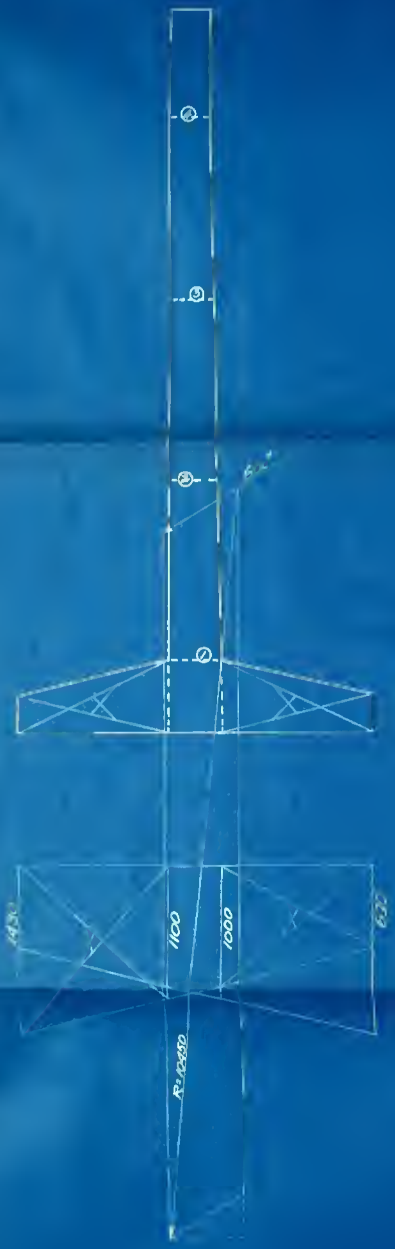
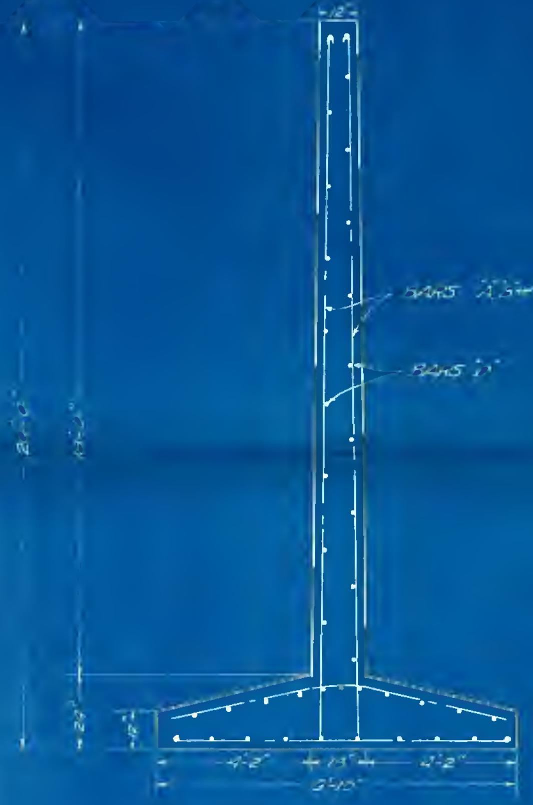


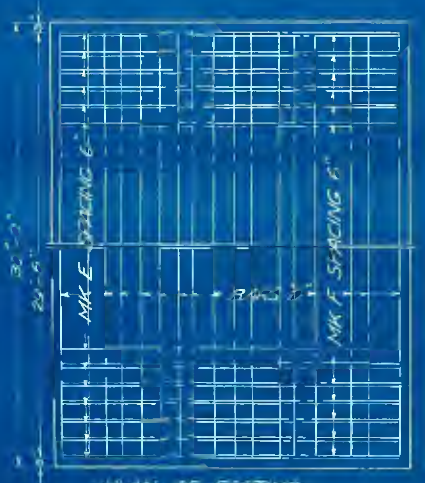
SECTION AT OPENING



CORNER BARS TO BE PLACED ON
ALL EXPOSED EDGES OF OPENINGS

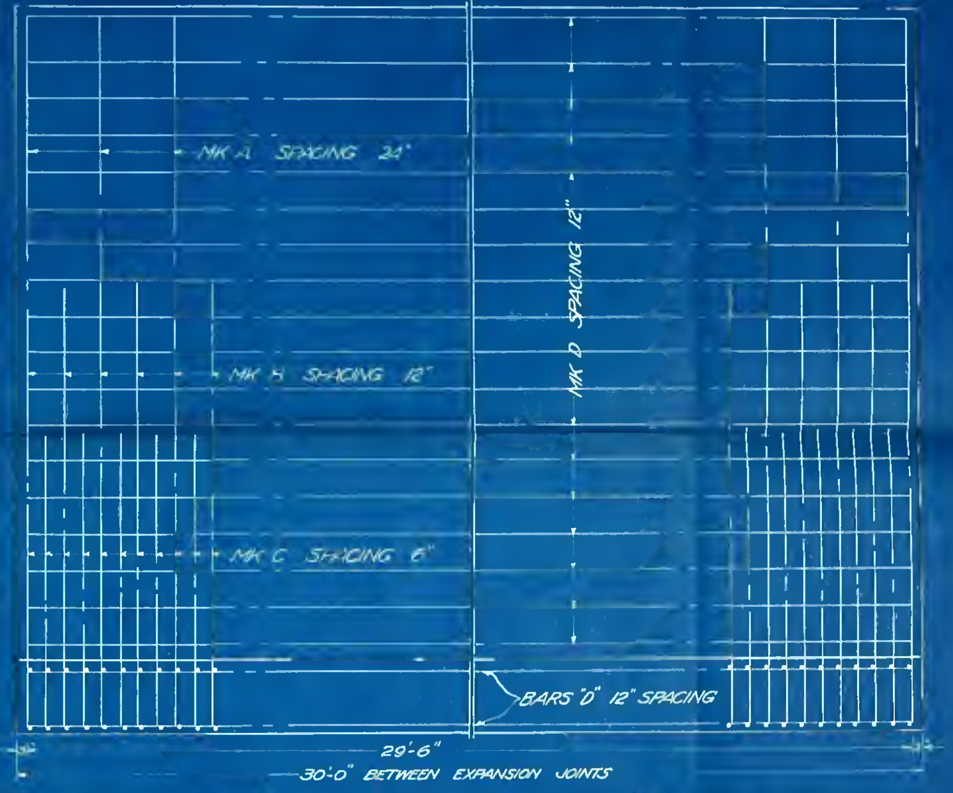






PLAN OF FOOTING

LONGITUDINAL SECTION



BILL OF MATERIAL FOR 30' SECTION

MARK	NO	SIZE OF BARS		REMARKS
A	32	3/4"⌀	19'-6"	
B	30	3/4"⌀	12'-0"	
C	60	3/4"⌀	7'-0"	
D	40	3/4"⌀	30'-0"	HOOKED
E	59	7/16"⌀	10'-0"	HOOKED
F	59	5/8"⌀	11'-0"	

NOTES
WT. OF CONCRETE - 150# PER CU. FT.
" " " " 36# " " "
NO SURCHARGE
FOUNDATION CLAY

BIN PARTITION WALL

DESIGN OF
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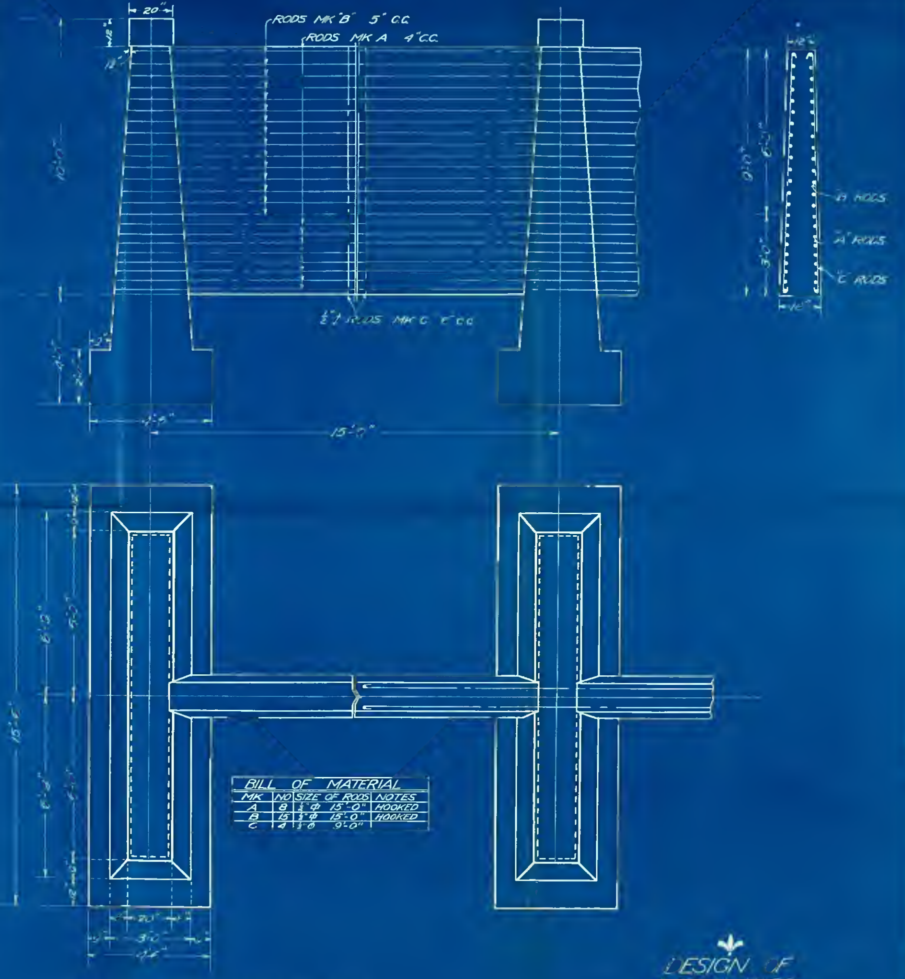
G. MOELLER
V.E. JOHNSON



BILL OF MATERIAL			
MK	NO	SIZE OF RODS	NOTES
A	8	½" Φ 15'-0"	HOOKED
B	15	½" Φ 15'-0"	HOOKED
C	4	½" Φ 9'-0"	

DESIGN OF WALL BETWEEN PIERS

DESIGN OF
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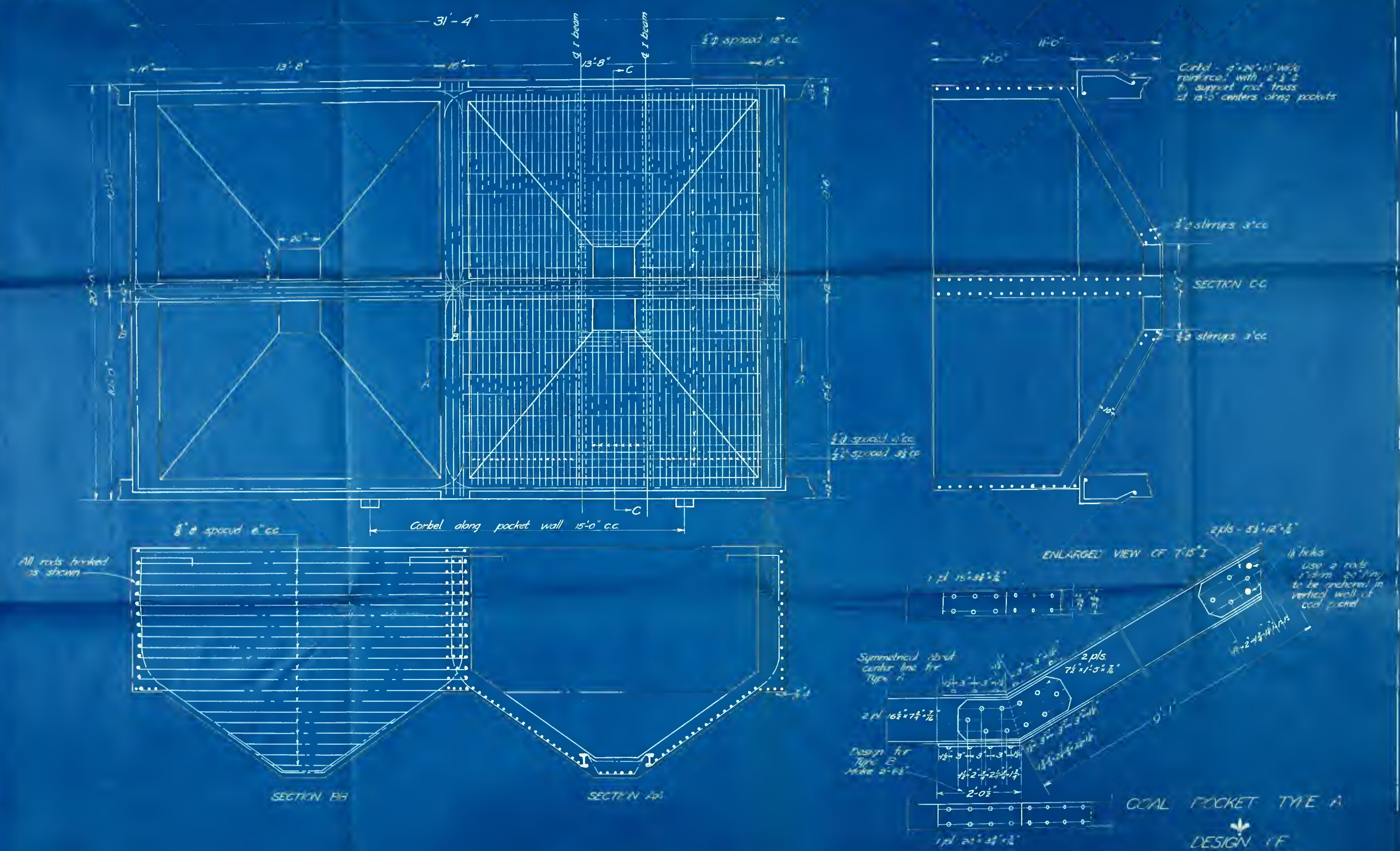

COAL POCKET TYPE A

DESIGN OF
A COAL YARD
OF
CONCRETE CONSTRUCTION

THESIS
MAY 1915
A.I.T.

F.N. CAMERON
A. MOELLER
V.E. JOHNSON

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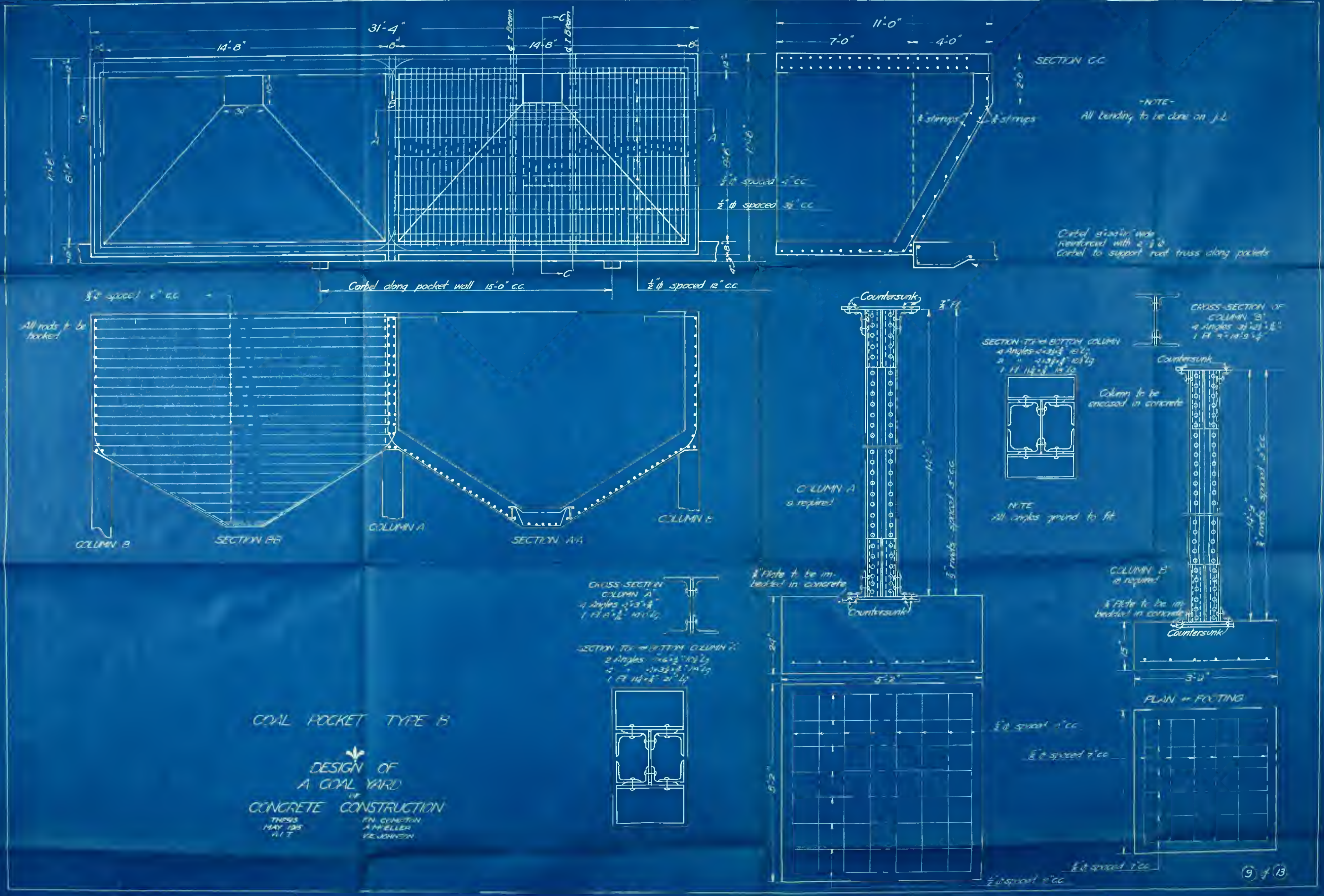

COAL POCKET TYPE B

DESIGN OF
A COAL YARD
OF
CONCRETE CONSTRUCTION

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MAY 1915
A.I.T.

F.N. COMPTON
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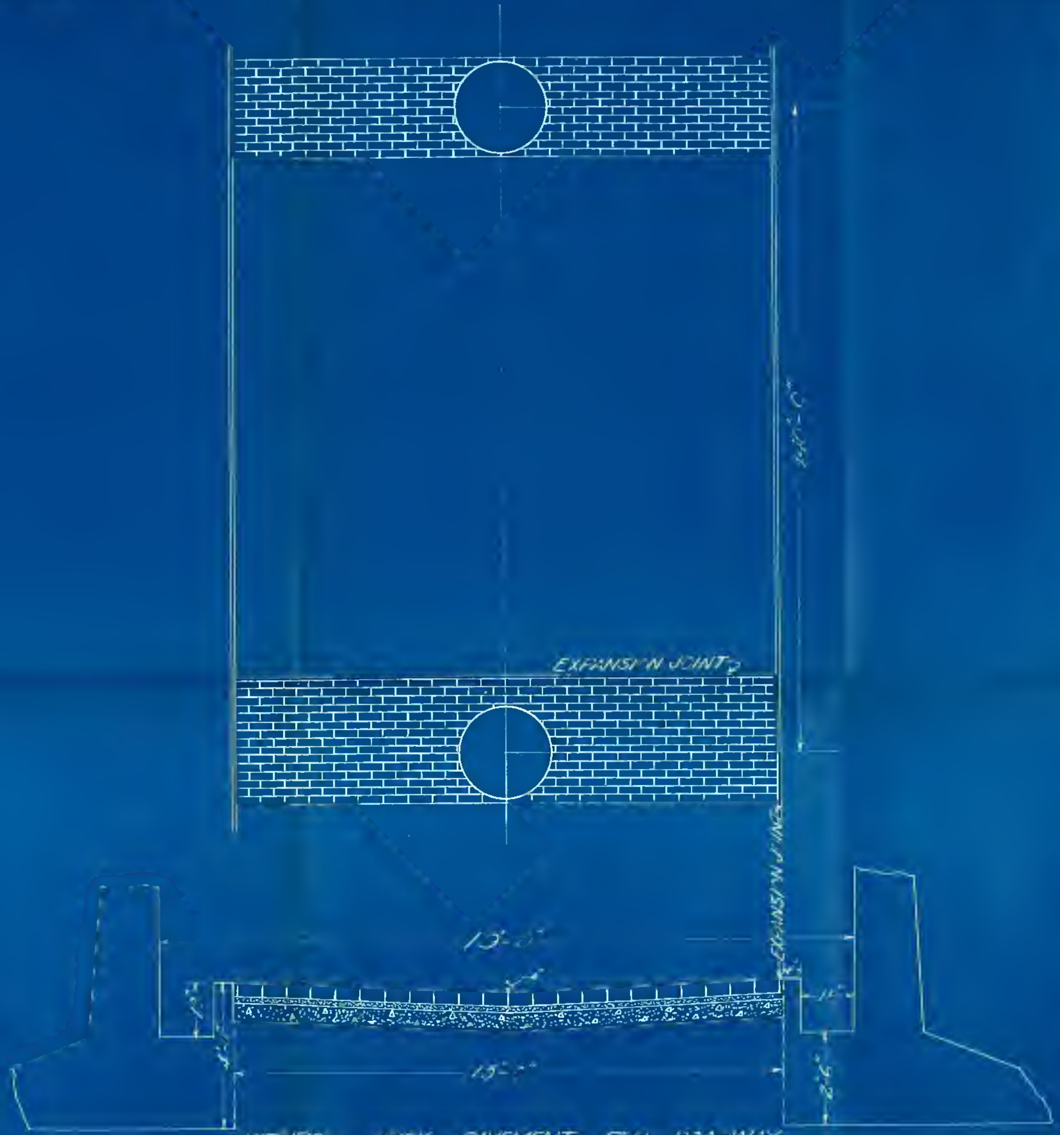

VITRIFIED BRICK PAVEMENT FOR ROADWAY

BRICKS 2½·4½·8"
SAND CUSHION 2" THICK
CONCRETE 6" THICK
EXPANSION JOINTS 1" WIDE

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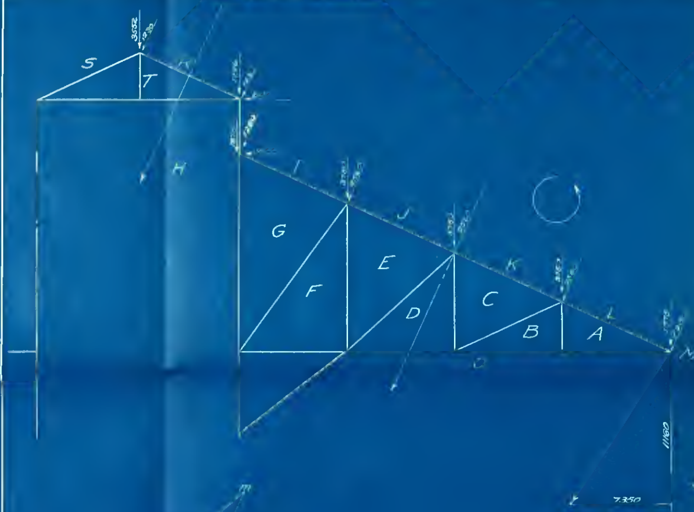
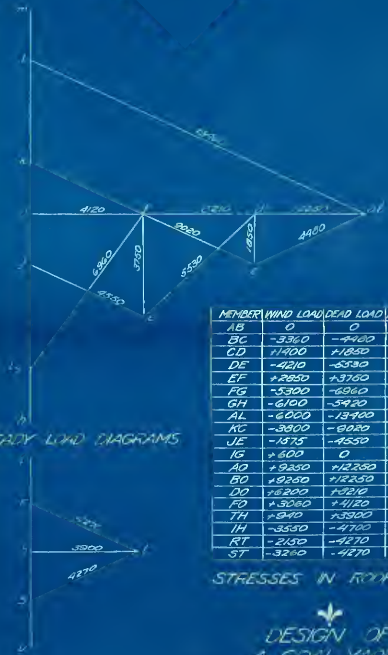
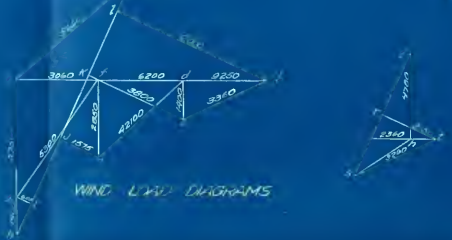




STEADY LOAD DIAGRAMS



WIND LOAD DIAGRAMS

MEMBER	WIND LOAD	DEAD LOAD	MAXIMUM
AB	0	0	0
BC	-3360	-4480	-7840
CD	+1400	+1850	+3250
DE	-4210	-5530	-9740
EF	+2850	+3750	+6600
FG	-5300	-6960	-12260
GH	-6100	-5420	-11520
AL	-6000	-13400	-19460
KC	-3800	-8020	-12820
JE	-1575	-4550	-6125
IG	+600	0	+600
AO	+9250	+12250	+21500
BO	+9250	+12250	+21500
DO	+6200	+8210	+14410
FO	+3060	+4120	+7180
TH	+940	+3900	+4840
IH	-3550	-4700	-8250
RT	-2150	-4270	-6420
ST	-3260	-4270	-7530

STRESSES IN ROOF TRUSS

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A. I. T.

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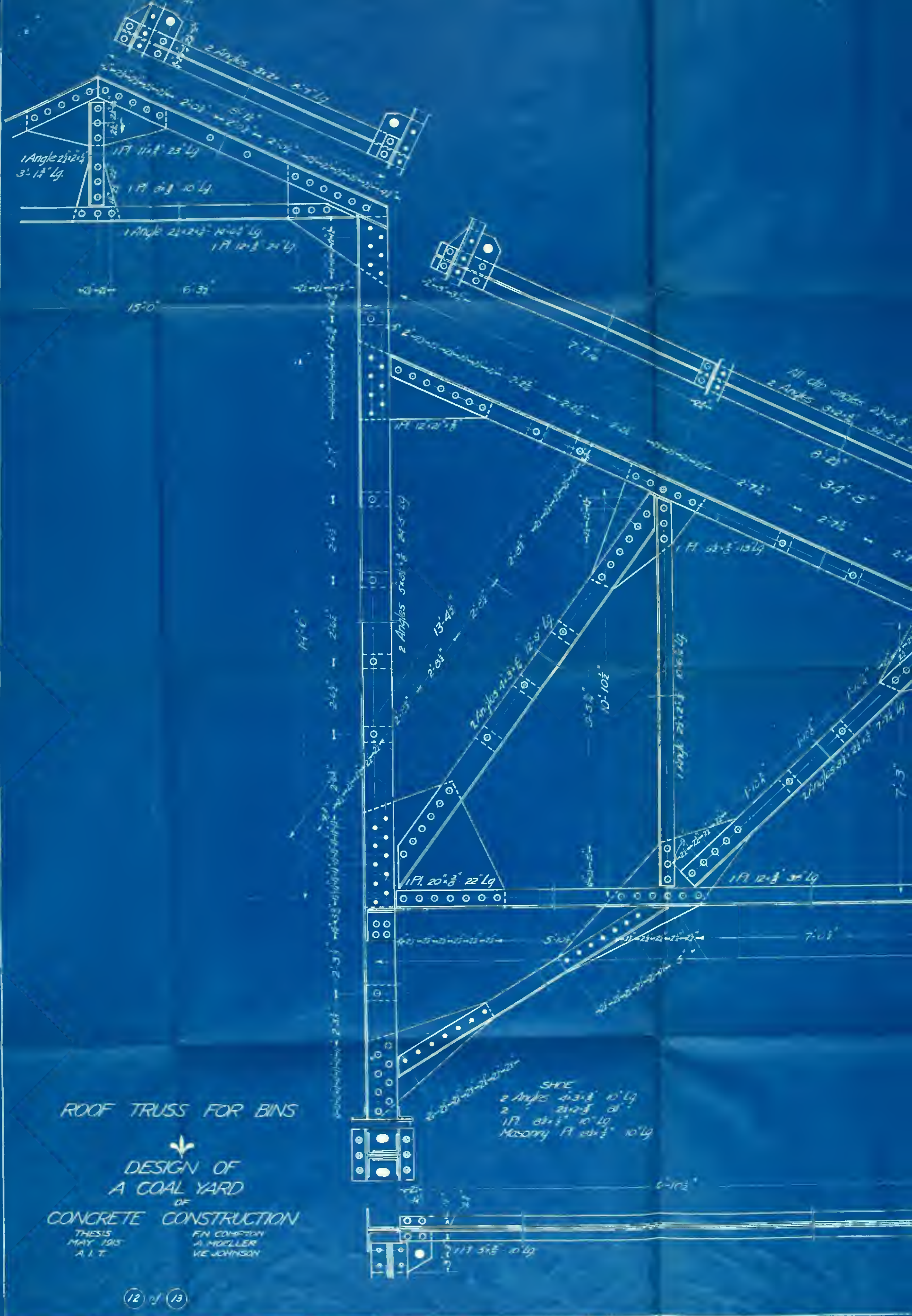

ROOF TRUSS FOR BINS

DESIGN OF
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CONCRETE CONSTRUCTION

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MAY 1915
A. I. T.

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A. MOELLER
V.E. JOHNSON

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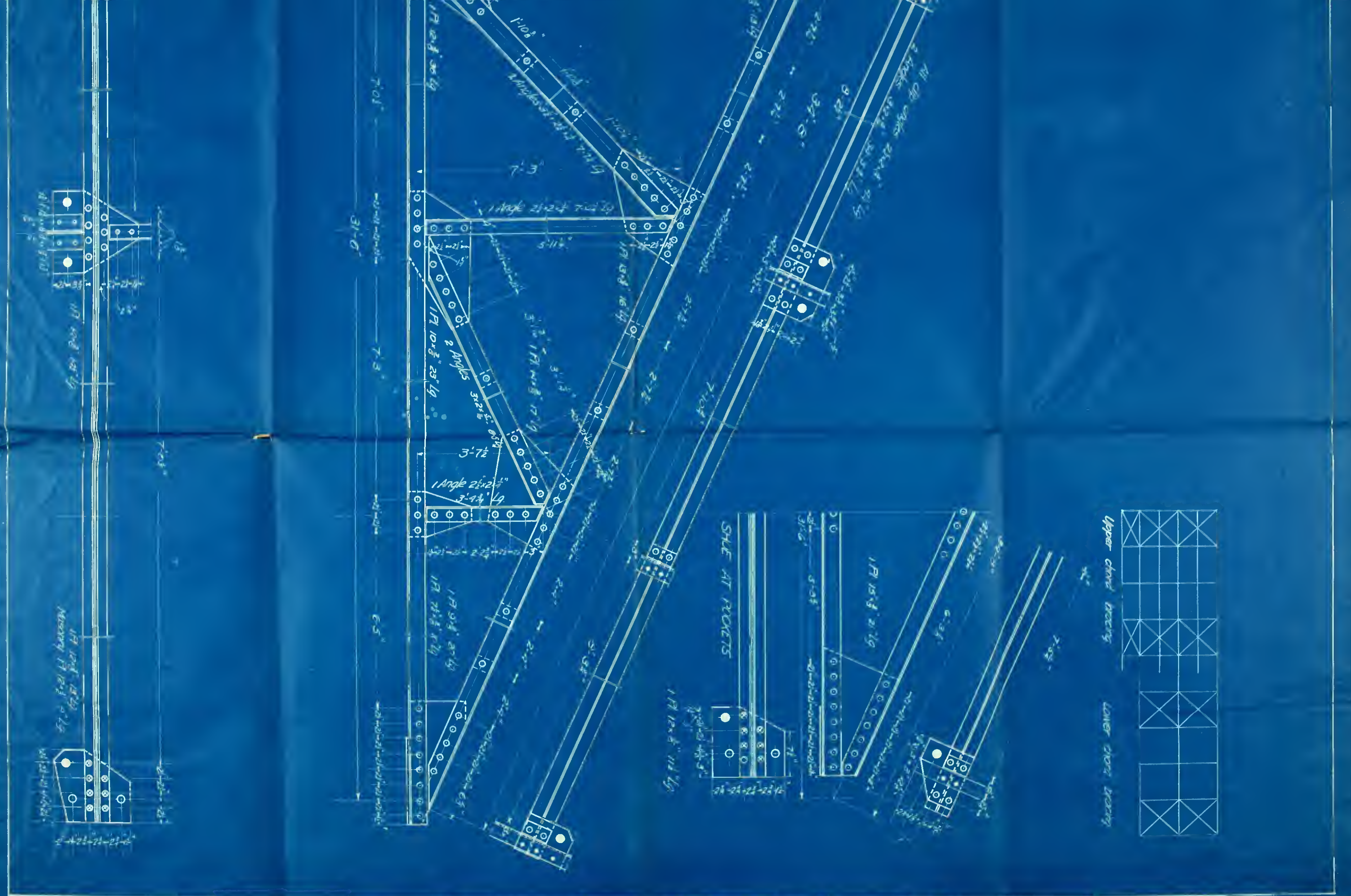
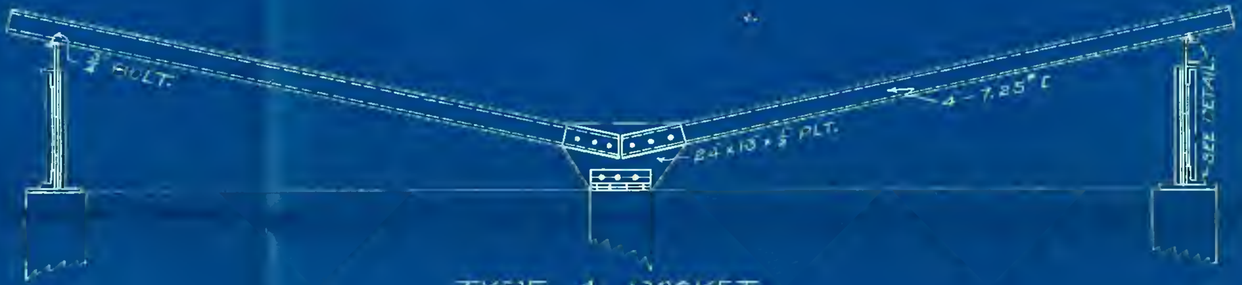
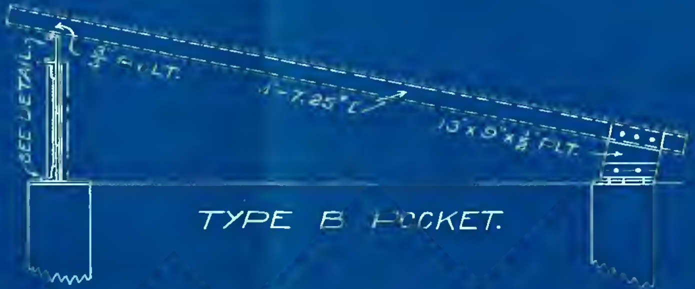

7'-3"
1 Angle 2½×2×¼ 7'-0 Lg
5'-11½"
1 Pl 10×⅜" 23" Lg
2 Angles
3'-7½"
1 Angle 2½×2×¼"
3'-4½" Lg
1'-10⅛"
SHOE AT POCKETS
Upper chord bracing
Lower chord bracing

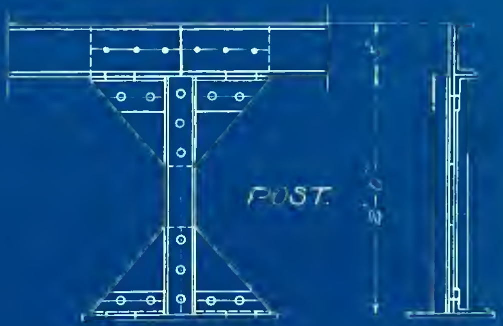
POCKET ROOFING CONSTRUCTION.



TYPE A POCKET.



TYPE B POCKET.



POST.

POST MATERIAL.

1 ⊏ 5"-9.00# - 15'-0"
2 ⊏s 3"-4.00# - 2'-6"
4 ∠s $3\frac{1}{2}$" × $2\frac{1}{2}$" × $\frac{1}{4}$" × 6'-7"
PLT. 20" × 9" × $\frac{1}{2}$"
" 15" × 9" × $\frac{1}{2}$"
" 18" × 15" × $\frac{1}{2}$"

DESIGN OF
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CONCRETE CONSTRUCTION.

—THESIS—
MAY 1915.
A.I.T.

F.N. COMPTON.
A. MOELLER.
V.E. JOHNSON.

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